HARBOUR ENGINEERING

BY

BRYSSON CUNNINGHAM, D.Sc.

A TREATISE ON THE

PRINCIPLES AND PRACTICE

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BRYSSON CUNNINGHAM,

D.Sc., B.E., F.R.S.E., M.Inst.C.E.,

EXHIBITIONER OF THE ROYAL UNIVERSITY OF IRELAND; MEDALLIST AND PRIZEMAN OF THE CITY AND GUILDS OF LONDON INSTITUTE;
AUTHOR OF "THE PRINCIPLES AND PRACTICE OF DOCK ENGINEERING,"
"THE DOCK AND HARBOUR ENGINEER'S REFERENCE BOOK,"
"A TEXT-BOOK ON BUILDING CONSTRUCTION," ETC.

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PREFACE TO THE SECOND EDITION.

Since the publication of the first edition of *Harbour Engineering* very considerable progress has been made in this field of Engineering Science, and the author has been engaged for some time in rewriting and making additions to the text and illustrations. It was found quite impossible to keep within the limits of the first edition, and the book has been enlarged by about 100 pages, including over 60 new figures. In this extension the author has had the cordial co-operation of the publishers, who desired, notwithstanding the costliness and scarcity of materials at the present time, to provide a volume which would do justice to the subject and be a worthy companion to *Dock Engineering*. War conditions have, of course, necessitated certain restrictions, and the book has been submitted to and censored by the Admiralty, so as to avoid publishing information which might prove useful to the enemy. Plans and descriptions of recent developments at Government Works and Naval Harbours have had to be omitted, but the authorities have been able to see their way to allow references to certain constructional processes and details.

The inevitable delay in the production of this edition has entailed the work being out of print for a few months, but it is hoped that in its new and extended form the volume will prove of greatly increased service.

The author's thanks are again due to several professional friends who have kindly read various sections of the text and favoured him with comments and suggestions.

London, *November*, 1917.

PREFACE TO THE FIRST EDITION.

THE favourable reception accorded to his treatise on *Dock Engineering* led the author to consider the possibility of supplementing that work by a companion volume on the kindred subject of *Harbour Engineering*.

The principal difficulty lay in making such a treatise complete and self-contained without recapitulating a good deal of material which had appeared in the earlier book, and which was equally essential in the present instance, the two departments of maritime engineering having so much in common as to be generally practised together.

The most acceptable solution which suggested itself consisted in treating this common material in a somewhat different way from that previously adopted, by presenting fresh points of view, additional features of interest, and new illustrations. This plan has been followed, and it is hoped that it will meet with approval.

Once more the author has to express his indebtedness to a number of personal and professional friends who have rendered him kind and valued assistance in the execution of his task; to the writers of papers and to the societies who have courteously allowed him to make extracts from the various minutes of proceedings alluded to in the text; and to the editors of *Engineering* and *The Engineer* for much useful information gleaned from the columns of their respective journals.

As in the case of *Dock Engineering*, every care has been taken in regard to the accuracy of data and statistics, but mistakes are always possible, and any intimation of their presence will be gratefully appreciated.

BRYSSON CUNNINGHAM.

LONDON,
January, 1908.

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The Ingersoll Sergeant Drill Co., London : Drills.
Nobel's Explosives Co., Glasgow : Explosives.
W. H. Bailey & Co., Manchester : Dynamometers.
Stothert & Pitt, Bath : Block-setting Plant.
Chance & Co., Birmingham : Lighthouses and Illuminating Apparatus.
Brown, Lenox & Co., London : Mooring Buoys.
Pintsch & Co., London : Channel Buoys and Lightships.
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A. Légé & Co., London : Automatic Tide Gauge Recorders.
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HARBOUR ENGINEERING.

CHAPTER I.

INTRODUCTORY.

Harbour Engineering and Navigation—Natural and Artificial Harbours—Ancient Sea Routes—Phœnician, Egyptian, Grecian, Carthaginian, and Roman Harbours—Mediæval Navigation—The Cinque Ports—The Hanseatic League—National Interest in Harbours—State Subvention.

Harbour Engineering and Navigation.—The history of harbour engineering runs concurrently, through corresponding stages from origin to development, with the history of navigation. Nor is the fact at all surprising. From the very nature of the case little else could be expected, since the two sciences stand to one another in the closest inter-relationship of cause and effect. With the appearance on the seas of the first craft calling for the exercise of expert seamanship, there arose a need of havens in which it might not only find shelter during stress of weather, but also take in and discharge its cargoes under suitable conditions. And as vessels gradually increased in number, size, and importance, so the need for more spacious accommodation became the more pressing, and the demand for larger and better harbours the more imperative.

Natural Harbours.—Of natural creeks and basins, possessing intrinsically all the advantages which a haven of safe anchorage requires, there are in the world not a few, and, no doubt, at the outset they abundantly sufficed for the rudimentary necessities of the early mariner. But the accommodation afforded was in many cases limited, and, as time elapsed, it became less and less compatible with the exigencies of rapidly expanding navies, whether engaged in commerce or war. Neither did the situation of these inlets always prove convenient, more especially to trading vessels. Some of the most commodious of them are to be found far out of the track of well-established lines of communication, and away from the principal routes of over-sea trade. And of those which are conveniently accessible, few, if any, have realised the ideal of a completely sheltered haven. There has been almost

invariably some inherent defect to be remedied, some deficiency to be made good. Accordingly, even the best of natural harbours have called for improvement and development, while, in the absence of natural advantages, steps have had to be taken to provide by artificial means the requisite degree of shelter and protection.

Artificial Harbours.—In every country, therefore, finding itself in the possession of a seaboard, and with any pretensions to maritime enterprise, there has been formed by degrees a series of artificial or semi-artificial enclosures, constructed at first somewhat crudely and informally, but later with full application of scientific method and technical skill.

There exists, indeed, no record on parchment, bronze, or stone to attest the date of the inception of the first artificial harbour. In the absence of any controverting evidence, the honour of creating the prototypes of modern maritime engineering undertakings of this kind is usually assigned to the public-spirited policy of monarchs of early Egyptian and Phœnician dynasties. Yet this ascription of priority is, after all, one more of inference and assumption than of definite knowledge, and there is reason to suspect that artificial harbours, in an embryonic form at any rate, are of much more ancient origin, dating back through the earlier civilisations of the remoter East to a period of time of which all historical traces have been lost, and concerning which it would, for that reason, be useless to inquire and idle to speculate.

Ancient Sea Routes.—We have no alternative but to confine our animadversions within the limits of fruitful historical research. It is known for a fact that both Egypt and Phœnicia possessed commercial navies, and that they carried on an elaborate system of trading operations. Their maritime traffic was not only characterised by regularity and importance, but it was also of a mutual nature, and the two countries were linked together by ties of common interest and advantage.

The sea trade of Western Asia and the contiguous portions of Africa was conducted in ancient times principally along two routes. The first of these led from the Phœnician ports, *via* Cyprus to Sicily and Malta, with an extension along the northern coast of Africa, finally reaching Tartessus in Spain, the site of which was probably near where Gibraltar now stands. The second sea route was from Ezion Geber at the head of the Gulf of Akabah, along the Red Sea, skirting the Southern coast of Arabia to Ophir at the mouth of the Indus, the "land of gold and precious stones."

Besides these two main routes, however, there were a number of subsidiary tracks intersecting one another in various directions. It has already been mentioned that close commercial relations existed between Tyre and Sidon and the deltaic ports of the Nile. Apart from these, there was considerable traffic which passed into the mouth of the Euphrates, both from the coasts of India and of Africa. Nor were the more adventurous spirits of the age restricted to beaten tracks. Right up to the Cassiterides (Scilly Isles) in the far West sailed the Cabots and Frobishers of that age, as also

they may have circumnavigated India and penetrated to Burmah and the confines of Siam in the East.

Phœnician Harbours.—Artificial works were indubitably in existence at both Tyre and Sidon. The former town stood on a peninsula, flanked on each side by a harbour formed of moles of loose or random rubble. Sidon possessed similar works of perhaps a little less extensive character. On the testimony of ancient historians, Tyre was a magnificent city and a flourishing port, with properly constructed quays, equipped with substantial warehouses, dating back between two and three thousand years prior to the commencement of the Christian era. The town underwent several vicissitudes in the course of its history, even to the extent of being destroyed by the princes of Assyria, and afterwards rebuilt. It fell finally at the hands of Alexander the Great, B.C. 332, and although the town of Sur still marks the site at the present day, scarcely a vestige of the glory of the ancient city remains, and the world-renowned harbours have sunk through successive stages of disrepair and decay to ruin.

Fig. 1.—Ancient Harbour of Alexandria.

The Harbour of Alexandria.—Remarkable as were the harbour works of Phœnicia, they were far outshone by the more elaborate undertakings at Alexandria, originated by the Conqueror of Tyre and brought to a successful conclusion under the first two Ptolemies about 200 B.C.

Immediately in front of the city lay the long, low island[1] of Pharos, which, in itself, constituted an admirable natural protection, and, opposite each end of this, two coastal promontories jutted out, almost completely enclosing a large area of water. These projections were extended by means of breakwaters, so that an excellent anchorage was obtained, well sheltered from the sea. A long narrow embankment, called the Heptastadium, constructed midway, divided the strait into two almost equal portions, that on the west being known as the Harbour of Eunostos, or the Happy Return, and that on the east as the Great Harbour. At each end of the Heptastadium was

[1] Now become part of the mainland.

a passage, spanned by a bridge, forming a connection between the two sheets of water.

The Great Harbour was entered from the Mediterranean through a channel, not altogether free from danger owing to its narrowness, between the island of Pharos and the breakwater built out from the Lochias Promontory. At this extremity of the island was the famous Pharos[1] Lighthouse or Tower, built by Sostratus of Cnidus during the reign of Ptolemy Philadelphus, 270 B.C. Its height has been variously stated; it was built of white marble and passed at the time for one of the "seven wonders of the world." At its summit a powerful beacon light was visible for a considerable distance.[2]

Grecian Harbours.—The Greeks and cognate races were notable harbour engineers, and their handiwork was made manifest at Rhodes, Salamis, Corinth, Syracuse, and many other places. Perhaps the most noteworthy instance was Piræus, the harbour of Athens, situated at the mouth of the Cephisus, about three miles distant from the capital city. It was a most capacious harbour, inclosing three large basins,[3] called Cantharos, Aphrodisium, and Zea. These were naturally formed, but were improved and extended by moles (490-480 B.C.), under the direction of Themistocles, so

[1] The title *Pharology*, applied to the science of lighthouse construction, is derived from the name of this tower.

[2] The height, and consequent range of visibility, of the Pharos Lighthouse has been a much debated question. Seyffert (*Dictionary of Classical Antiquities*) quotes Josephus as comparing it with the tower of Phasael at Jerusalem, which was about 90 cubits, or 135 feet in height. Weigall (*Life and Times of Cleopatra*) says it rose to a height of 400 ells, or 590 feet. Edrisi made it 300 cubits, and Stephanus of Byzantium, 306 orgyiai, or about 1,836 feet (Prof. Middleton). Weigall gives the authority of Josephus for a visible range of 34 miles. Lemprière (*Classical Dictionary*) advances the allegation that it was visible for a distance of 100 miles. As a strict statement of fact, this is perhaps hardly credible, but, by reflection in the clouds, the light may have been observed at a greater distance than the mere geographical range due to the height of the tower.

[3] The location of these basins is a matter of some uncertainty. The arrangement in fig. 2, due to Leake, is that perhaps most commonly accepted, but Dr. Ulrichs (*Topo-*

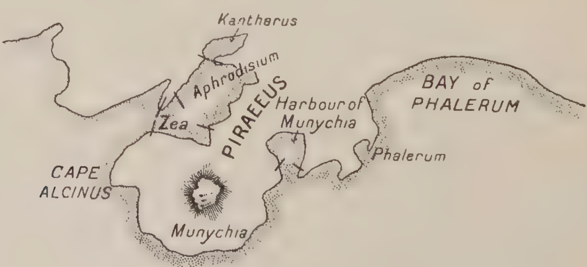

Fig. 2.—Ancient Harbour of Athens.

graphy of the Harbours of Athens) places the harbour Zea on the eastern side of the peninsula and identifies it with the inlet marked Harbour of Munychia, which latter harbour he places further east (Phalerum).

that they became sufficiently commodious for the reception of a fleet of 400 ships.

Harbour at Carthage.—The Carthaginians, as might be expected from their blood relationship to the parent stock of the Phœnicians, also developed a talent for harbour construction, and they made Carthage a model port. It comprised two compartments inclosed by breakwaters and connected by a channel 70 feet in width. Around the inner basin, which afforded space for over 200 ships of war, were located the arsenals and stores. When Scipio blockaded the place in B.C. 146, cutting off communication with the sea by means of a dam across the entrance to the outer harbour, it is recorded that the Carthaginians, with characteristic energy, excavated for themselves a new outlet. Their exertions and strenuous defence were, however, without avail, and the downfall of the city took place shortly afterwards.

Roman Harbours.—At the close of the second Punic War and considerably prior to the event just narrated, the world sea-power had passed into the hands of the Romans, and with it the genius and capacity for harbour construction. Ostia, Ancona, Antium, and Civita Vecchia, amongst hundreds of other instances, may be cited as evidences of this fact. The works carried out by the Latin race were of a solid and enduring character, which in many cases have defied alike the ravages of time, storm, and devastation. Civita Vecchia still possesses a serviceable harbour capable of receiving vessels of 20 feet draught. The works at Ostia also exist, although the town, formerly at the mouth of the Tiber, is now twenty miles or so inland.

Mediæval Navigation.—Passing to mediæval times, we find a vast expansion in maritime trade and a corresponding increase in the number and size of harbours. The whole of Europe was now engaged in avocations connected with the sea and embarking on nautical enterprises as adventurous as they were remunerative.

This is not the place, however, in which to attempt anything of the nature of a historical and analytical disquisition on the growth and expansion of seaborne commerce, nor even is there space to describe the provision made for its reception and accommodation at the various ports with which it was associated. We do not propose, therefore, to dwell further upon this part of the subject beyond making a brief allusion to two features of outstanding interest and importance, showing how closely the commercial and political welfare of a maritime country is bound up in the maintenance and development of its seaports and harbours.

The Cinque Ports.—The first of these features is the formation of a confederation, of which only a name and an office, and that a pure sinecure, remain at the present day. The Cinque Ports were, and for that matter are, as the name implies, a group of five ports in this country fronting the English Channel. The towns were originally Hastings, Romney, Hythe, Dover, and Sandwich, and subsequently there were added—Winchelsea, Rye, and Seaford.

They represented the naval activity of this country, and they were responsible for the protection of the Kentish coast against the incursions of foreign foes. To this end they held certain levies of shipping constantly at the disposal of the crown, and, in return, they had conferred upon them several special distinctions and privileges.

At this distance of time, it is difficult (with, perhaps, one notable exception) to think of these insignificant villages as forming the forefront and backbone of England's naval power. Yet from the modest moorings and lowly quays of these Kentish harbours slipped away many a valiant little cog to confront the caravels of France and the galleons of Spain. But more than this, they were, indeed, the very seed and nucleus of England's foreign trade: inferior, certainly, to London in importance, but, during their palmier days, vying with Bristol and Plymouth in the west in the honour and distinction of seaward enterprise, and forming the principal links in the chain connecting England with the Continent and with all the commercial products of the civilised world known at that epoch.

The Hanseatic League.—The other noteworthy feature was the Hanseatic League. This was an association of German cities inaugurated about the twelfth century, or perhaps earlier (for the real origin of the association is involved in some obscurity), for the protection and advancement of seaborne commerce generally, and more particularly to foster their own interests therein. The combination grew in importance and became ultimately exceedingly influential, embracing a number of ports in the Netherlands, France, Spain, and Italy, and also London in this country. For a considerable time the League enjoyed such power as to render it well-nigh independent of national jurisdiction, but gradually, by absorption and suppression, its privileges were curtailed, until they practically disappeared towards the close of the seventeenth century. What now remains of the confederacy is strictly limited to the three German ports of Hamburg, Bremen, and Lubeck. But during the period of its greatest glory and power, it exercised a far-reaching influence in the encouragement and development of trade both by land and sea, and especially in regard to the administration of port dues and charges.

These two historical episodes illustrate in a very marked degree the close inter-relationship of national policy and commercial enterprise, and they demonstrate how essential to the prosperity of maritime nations is the maintenance and protection of their seaports. There are few countries in the world which are so unfortunate as to possess no seaboard. What few there are, are insignificant in size and in political importance. It is the definite aim and object of most countries, where possible, to increase the extent of their sea frontage. More than one modern war has been really, if not ostensibly, due to the endeavour of a nationality handicapped by a restricted littoral to attain improved communication with the open sea, or, in some cases, even to gain simply direct access to it. The sea is the great highway

of the world, a spacious and practically limitless expanse whereon transport is a process at once simple, economical, and direct.

National Interest in Harbours.—Such being the case, the inquiry can scarcely fail to arise: How far is the state responsible for the upkeep and development of its ports? Ought harbours to be under the control and tutelage of the nation, and, if so, what kind of patronage and protection, and how much of it, should the latter accord? Stated in concrete terms, should harbours be kept in a state of efficiency, not merely by means of local resources, but by direct governmental assistance, involving the contributions of inland towns? The question is a complex one, and admits of more than one answer.

In so far as a state is a naval power, it has absolute need for shelter and coaling places for its vessels of war. It is, therefore, without any question, entirely concerned in the provision and management of such depôts as are necessary for the purpose. Moreover, in states possessing a littoral frontier swept by fierce gales, it is also a matter of national expediency to produce at certain points works of a protective nature, which will enable imperilled shipping to survive the disastrous effects of sudden tempests.

So far the matter incontrovertibly affects the national welfare. Next, as regards interests which are open to the charge of being purely, or mainly, local.

In regard to ports which have grown up entirely on a commercial basis associated with markets and industries in the immediate vicinity, the same requisition for state interference is not so apparent, and while conditions are favourable and trade prosperous, there is little desire or need to raise the point for discussion. So long as local rates and charges are sufficient to meet all demands entailed in the upkeep of such commercial ports, it is difficult to see why they should not be allowed and encouraged to retain their own independence and work out their own programmes of development. It occasionally happens, however, that a commercial port falls behind the times; it may be from various causes—possibly from indifferent administration, mismanagement, culpable malversation, and so on, but generally and ultimately from lack of funds to carry out improvements which have become necessary by reason of the continually rising standards of shipbuilding. In such cases more than temporary stagnation is threatened. Maritime engineering works, having once become obsolete, cease to be utilisable at all in any practical sense, and there is no prospect before them of anything but a speedy decline of their trade. Now the state can hardly regard with equanimity the extinction of any one of its centres of commercial activity. Therefore it becomes a question—and the plea has been urged in at least one prominent instance of late—whether the state is not bound to step in with the necessary financial assistance or guarantorship, on the ground that by so doing she is favouring the interests of the community at large. In a general sense the contention is legitimate, but the application of such

a principle must necessarily be governed very largely by the special conditions of each particular case, since there may be circumstances under which a grant would be injudicious as well as unjustifiable.[1]

Lastly, in regard to the great majority of harbours—small, almost insignificant havens, many of them—fringing the coasts of every civilised country, where a hardy race of fishermen wring a strenuous and ofttimes scanty harvest from the sea. The districts in many cases are poor, and the calling could hardly be classified as lucrative. Yet there are many thousands of people in this country alone dependent on it for sustenance, either directly or indirectly. Where local resources are so utterly inadequate to cope with the difficulties of the situation, it is obviously impossible to look to them for the requisite outlay, and the principle of state subvention is now, indeed, fully recognised by the Board of Trade in all instances where little local communities are desirous of extending their accommodation within approved limits.

In making these grants [2] it has been stated that "it is regarded as essential that of the total cost required for construction, at least two-thirds should be provided from local or outside sources, and that the contribution from the exchequer should in no case exceed the remaining third." The action of the Committee has been limited "to the case of harbours serving, or likely to develop, a large fishing district either as points of departure and landing for the fleet, or as providing refuge on parts of the coast, where the nearest existing harbour is so distant as to destroy the value of fishing grounds which produce a good harvest of fish."

[1] "It may be urged that the expenditure involved in keeping ahead of the developments in shipping is greater than port authorities should be called upon to incur from their own resources, and there is doubtless, in some cases, something to be said in favour of that view, although I hold that such expenditure is reproductive in a variety of ways beyond the mere income arising from the exaction of dues. In my opinion, however, when port authorities, who have striven to provide and have provided certain facilities, are unable to incur the necessary expenditure for further development, it is desirable that the state should come to their assistance and thereby aid these authorities in developing ports on national rather than on local lines."—The Rt. Hon. Lord Pirrie on Harbour and Dock Requirements, Engineering Conference, 1907.

[2] The following are some of the more important grants which had been sanctioned up to a recent date :—

Pwllheli, Carnarvonshire,	£29,875
Fraserburgh, Aberdeenshire,	40,000
Peterhead, Aberdeenshire,	28,000
Lerwick, Shetland Islands,	4,500
Wick, Caithness,	20,000
Southwold, Suffolk,	25,000
Whitby, Yorkshire,	24,400
Scarborough, Yorkshire,	4,750
Brixham, Devonshire,	21,750
Buckie, Banffshire,	33,000
Newlyn, Cornwall,	11,000
Padstow, Cornwall,	6,650

Taking the British Isles as a whole, there are something like 130 to 140 of these fishing centres distributed among Ireland, England (including Wales) and Scotland respectively, very closely in the proportion of 1, 2, 3. These harbours provide sheltered areas of water ranging from 2 or 3 acres upwards, though a very large proportion of them are under 10 acres. They are, therefore, individually small, but as already stated, since no inconsiderable proportion of the population derive their livelihood from connection with them, their importance is not to be gauged by size alone.

The commercial ports of this country are less numerous. They number about a score, and the accommodation they provide is largely in the form of docks and inclosed basins of considerable area, both individually and collectively.

National harbours of refuge and for naval purposes are still fewer in number. The areas inclosed, however, are correspondingly larger and attain to as much as 500 and 600 acres a-piece, and even more.

Here our subject leads us on from general observations to an organised investigation of the principles of harbour design, which we can deal with to better effect in another chapter.

CHAPTER II.

HARBOUR DESIGN.

Difficulties of the Subject—Classification—Definitions—Roadstead—Harbour—Basin—Dock—Harbours of Refuge—Commercial Harbours—Fishery Harbours—Localisation—Coastal and Inland Ports—Procedure in Design of Harbours—Preliminary Considerations—Natural Phenomena—Prevalence and Intensity of Storms—Coastal Change—Accretion and Denudation—Effect of Artificial Interference—Influence of Effluents—Island Harbours—Harbour Areas and Entrance Widths—Illustrations of Harbours at Zeebrugge, Queenstown, Sandy Bay, Sunderland, Peterhead, Libau, Madras, Aberdeen, Whitby, and elsewhere.

Difficulties of Systematic Treatment.—That maritime engineering is a science of much complexity and no little incertitude, is but a trite remark to make. It will be admitted, without any controversy, that its operations are of necessity founded largely upon assumption and carried out by tentative rather than confident measures. Hypothesis, analogy, and experiment constitute its working basis, alike in regard to theory as to practice, to design as to execution. The whole field of it is beset by many and peculiar difficulties, and scarcely any other department of constructive work finds so many hazards and obstacles in the way of satisfactory accomplishment. The task of the engineer who sets himself to contend with the almost bewildering array of antagonistic forces incidental to maritime operations, is exacting in the extreme. The data upon which his calculations must perforce be based are often defective and their origin obscure. He has to deal with agencies not only conflicting but frequently also co-operative, and as destructive as they are capricious. His work is subjected to the most trying of all ordeals, in that it is constantly exposed to the risk of unascertainable possibilities. Occasions arise when the profoundest sagacity and the ripest experience may well prove to be at fault. Laws which hold good in one locality seemingly reverse themselves in another. The success of certain dispositions in one case is no guarantee of their efficacy elsewhere, still less justification for their general application. Each place has its own definite characteristics, its peculiar defects, and its special advantages, differentiating it from all other places. There is no uniformity, and very little similarity. Generalisation, therefore, is impossible, and classification becomes difficult.[1]

[1] In confirmation that this is by no means an exaggerated statement of the difficulties besetting the practice of harbour engineering, it is permissible perhaps to quote the following extract from the Presidential Address of Sir Maurice Fitzmaurice, C.M.G.,

Yet, in spite of these deterrent considerations, it is manifest that some system of treatment must be adopted, unless the principles of harbour engineering are to rest on a haphazard, heterogeneous basis, contrary to the spirit of all scientific procedure. Our endeavour, therefore, in the following pages will be to collate such data as are definitely acceptable, to elucidate as far as possible those problems which present themselves within the range of ordinary experience, and to lay down certain rules which may serve for general guidance to those engaged in adapting some of the most variable forces in nature to the use and service of man.

A clear conception of our purview is essential, so we must commence with one or two definitions.

Classification.—A **harbour** is primarily a place of rest and refuge—a place where safety and hospitality are to be found. But round this central idea have grown several accretions of meaning, the gradations of which it is desirable to point out.

Limiting our references, as is natural and proper, to the domain of navigation, we may observe that a vessel seeking shelter under stress of wind and weather may possibly obtain it as follows :—

(1) Within a tract of water, not necessarily inclosed in any way, even partially, but adjacent to or not far distant from the coast-line, where there is good holding-ground for anchors and some protection from the onset of heavy seas. Such conditions constitute a **Roadstead.** Roadsteads may be either natural or artificial. In the case of a natural roadstead, a deep channel,

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to the Institution of Civil Engineers in 1916 :—" In the case, for instance, of a harbour constructed on an open coast, where it is often necessary to enclose a large area of the sea, there are many questions the solution of which can only be dealt with as the work proceeds. It may be quite possible to definitely design the character of the works enclosing the harbour, to roughly calculate the velocity of the tidal water entering or leaving the harbour through an opening or openings of certain widths ; but in many cases it is extremely difficult to judge what currents will be set up outside the harbour by the intrusion into the sea of such works, or the amount of deposit which may be expected in the harbour. A careful survey of the coast and sea-bed for a considerable distance on each side of the proposed works, a study of all existing currents and tidal effects in the locality, the collection of information from pilots and others who are accustomed to the navigation of the coast, the investigation of what has happened in any adjacent harbour or one situated in a similar position, are all of great help. After all this has been done, there are, however, very few engineers who, when there is a considerable range of tide, would care to state the character or velocity of new currents which may be formed near the entrances, or the amount of deposit which may take place in the harbour. Naturally, it is advisable to keep openings as small as possible consistent with safe and easy navigation, so as to reduce wave disturbance in the harbour, and an engineer will, therefore, probably not fix the width of openings, nor the exact position of the entrance works, until information about currents has developed as the work proceeds. Again, in harbour works on some coasts where sandbanks vary from time to time, few engineers find that works can be carried out exactly as originally designed. It is nearly always found that as the works proceed very considerable variations occur in the positions of shoals and channels, which necessitate modification in design.

with an intervening bank or shoal to seaward, possesses the necessary characteristics, as exemplified in the offing of the Port of Ostend (fig. 3).

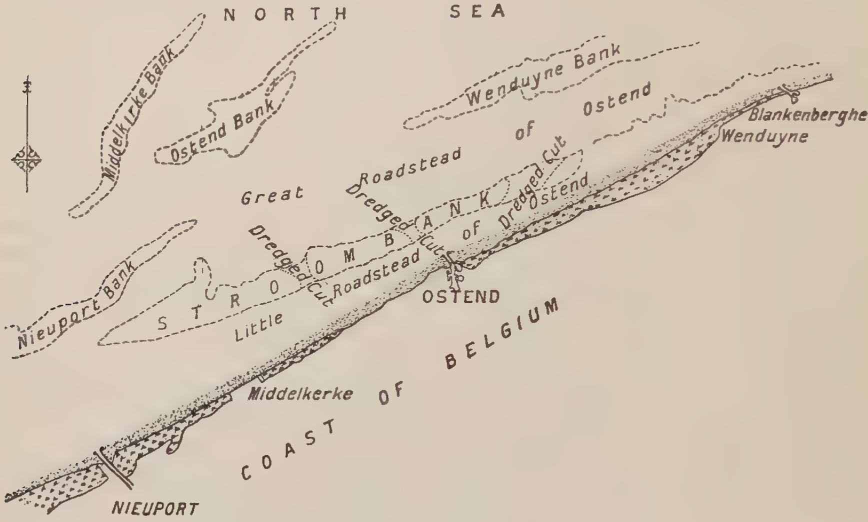


Fig. 3.—Roadsteads of Ostend.

An artificial roadstead may be created on similar lines by a breakwater, either parallel to the coast, or curvilinear, such as that at Zeebrugge (fig. 4).

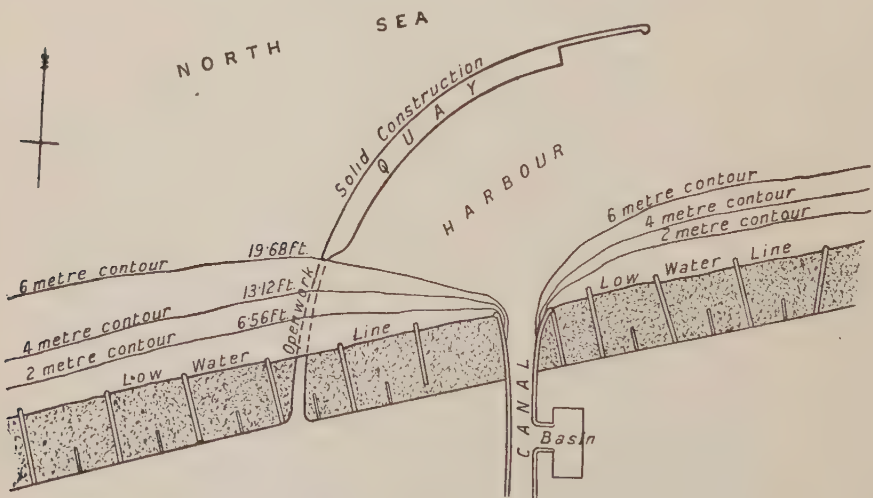


Fig. 4.—Zeebrugge Harbour.

(2) Within a definitely circumscribed area, almost completely inclosed, either naturally, as in the case of a creek or estuary, or artificially, by projecting piers, moles, and jetties. The harbour of Queenstown (fig. 5) is an

exemplification on a large scale of the former class, while the outstanding piers at the entrance to the River Wear (fig. 6) are typical of the latter, as

Fig. 5.—Queenstown Harbour.

also are the breakwaters at Portland (fig. 7) and Peterhead (fig. 8). Most harbours of importance fall under this category.

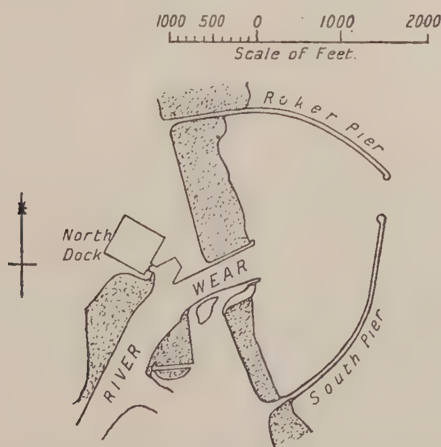


Fig. 6.—Sunderland Harbour.

(3) Within a confined **Basin** of comparatively small extent, having a narrow aperture only for the ingress and egress of vessels. There is little

to distinguish this from what is termed a **Dock,** though the latter expression is commonly restricted to basins provided with entrance gates. An illustration of this class of harbour is to be found at Peterhead in the South Harbour (fig. 8), while a dock at Sunderland (North Dock) is shown in fig. 6. Fishery harbours generally belong to this class, and it is no uncommon feature to find a smaller inner harbour, or basin, constructed in conjunction with a large outer harbour, or a roadstead. These basins are provided with quays for the reception of cargoes.

For the purpose at present in view, it will be found advantageous to

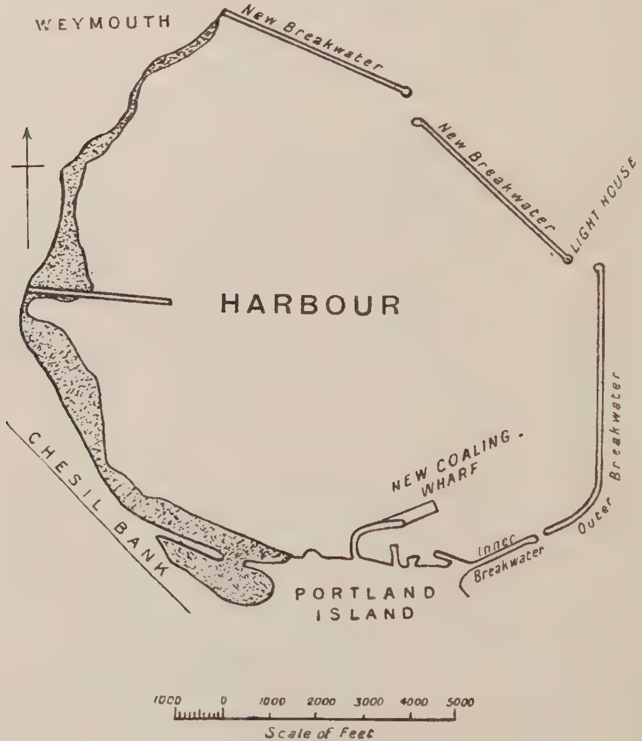


Fig. 7.—Portland Harbour.

adopt a slightly different classification based on the object to be attained. From this standpoint there are three important divisions, as follows :—

(1) Harbours of Refuge, including Naval Harbours and Bases.
(2) Commercial Harbours, associated with Ports.
(3) Fishery Harbours.

Though fundamentally in unison, and oftentimes found in combination, the designs of these three classes are sufficiently distinct to justify us in treating them separately, describing their particular functions and enumerating their special requirements.

Harbours of Refuge.—The principal duty of a Harbour of Refuge is, as the name implies, to provide a refuge for vessels overtaken by sudden stress of weather, or otherwise hard pressed or disabled. The proper *locale* for the construction of such harbours is obviously at conveniently accessible stations upon coasts which are inhospitable and dangerous. Yet, manifest as is the desideratum, the means of its accomplishment is not so obvious, and the subject has given rise to some conflict of opinion. Is the proper position for a harbour of refuge upon an outstanding frontage or within a bay? Ought it to be projected into the open, or recessed within the coast-line? In the former case, the goal is more easily reached and less delay is incurred in putting out again to sea; on the other hand, there is greater exposure, and

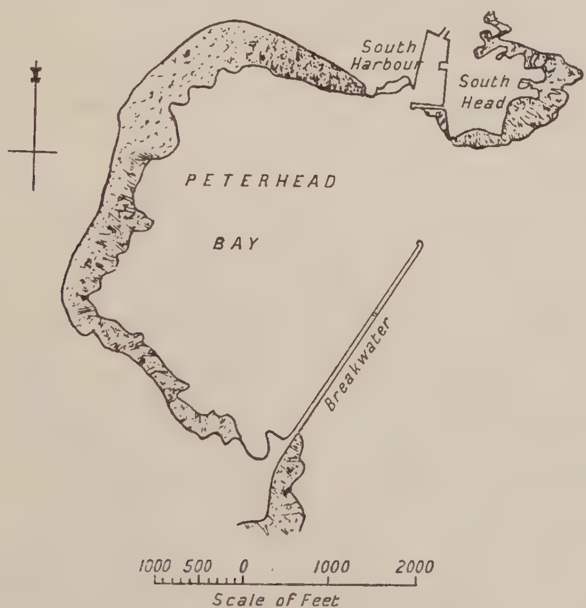


Fig. 8.—Peterhead Harbour of Refuge.

this endangers the ingress of vessels, rendering them more liable to miss the entrance, in which case they will probably be driven on to the shore. Yet the risk of ultimate catastrophe must necessarily be greater in the case of a vessel missing the entrance of an embayed harbour. It is, therefore, scarcely safe to dogmatise upon the point. There may be advanced positions to which a lofty headland imparts all the advantages of a sheltered recess, and there are likewise cases in which deep coastal indentations afford very meagre protection from tempestuous seas. Where it can be assured without serious risk, the nearer the haven to the distressed ship, the better her chances of reaching it. Many seamen, however, prefer, where practicable, to ride at anchor in the open rather than make for the uncertainties of the shore. The attainment of harbourage does not necessarily imply that the vessel is beyond

the reach of harm. Even in harbours provided with breakwaters, more particularly in cyclonic areas, there is an element of risk to certain classes of shipping in a heavy gale, and under certain conditions at such harbours vessels capable of doing so are advised to leave their moorings and put to sea on the approach of a hurricane.[1] Storms of such magnitude, however, are rarely, if ever, experienced in European waters, and the shelter provided in harbours of refuge on the British coast effectually disposes in this case of the necessity for any such action.

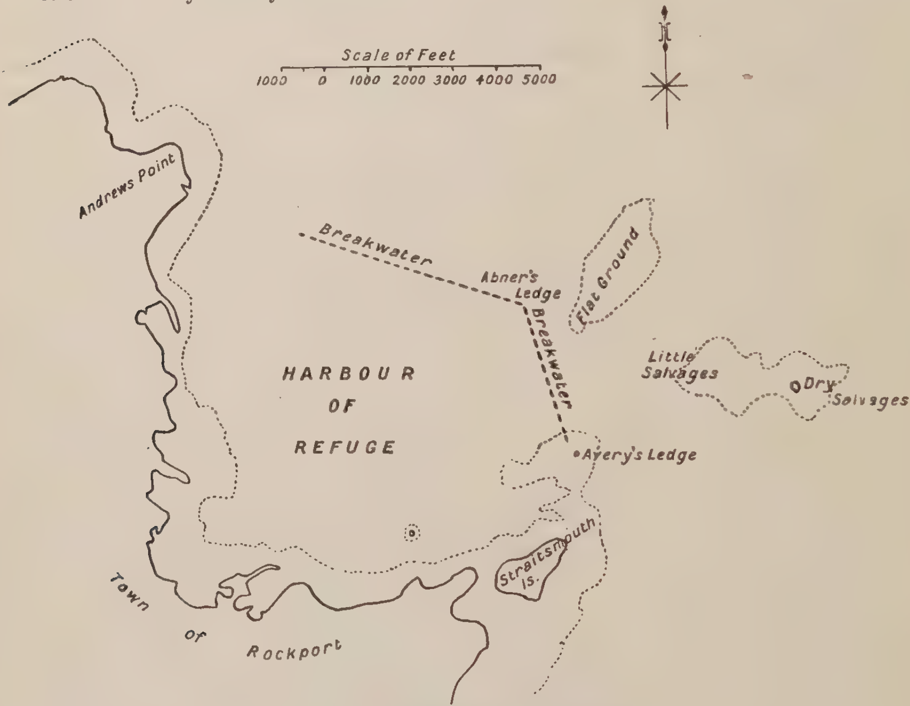


Fig. 9.—Harbour of Refuge, Sandy Bay, Mass., U.S.A.

The requirements of a harbour of refuge may be summed up as three :—

(*a*) Ready accessibility.

(*b*) Safe and commodious anchorage.

(*c*) Facilities for obtaining supplies and for executing minor repairs.

Accessibility.—Upon the first point we have already dwelt a little. But accessibility depends not only upon the site of the harbour ; it depends also upon its disposition. The entrance must be conveniently placed and designed, so as to allow of its being easily taken by ships driving before a storm. A

[1] At Madras, for instance, commanders are recommended by the port authorities to put out to sea as soon as the "great danger" signal is exhibited on the port flagstaff. Madras, of course, is primarily a commercial harbour, not a harbour of refuge, and the illustration is only used in justification of a sailor's preference for the open in tempestuous weather.

narrow entrance is difficult to negotiate, but, on the other hand, a wide entrance exposes the interior to the effects of rolling seas. Local circumstances will largely influence the determination of the dimension to be accorded thereto; at the same time, it may be said that from 600 to 800 feet approximately represents the expression of modern British practice. It is not unusual to provide more than one entrance; sometimes two in opposite directions, so as to afford a choice of approaches to vessels under varying conditions of wind. An entrance may also be deflected in order to afford cover to the interior. Provided a vessel is able to reach the shelter of an outer breakwater, it becomes more amenable to control, and it may be navigated into an inner berth with much less risk and trouble. An entrance may, accordingly, be placed to receive ships direct from the most exposed quarter, while at the same time the channel, or passage, may be diverted towards the interior, in such a way as to mitigate the influx of rough seas. A roadstead forms a useful vestibule to a harbour in this respect.

Size of Harbours.—In connection with harbours of refuge and naval harbours, it is appropriate to consider the question of size for important harbours generally. Naturally the determination of the matter largely depends upon the maximum number of ships to be accommodated simultaneously, and to a certain degree on the size of the largest vessel likely to use the harbour. The rapid growth of modern ships, both in the Royal Navy and in the Mercantile Marine, makes it a point of fundamental prudence to allow ample space for developments. The following tables show the rate of increase in the size of ships within recent years :—

LENGTH AND TONNAGE OF LARGEST COMMERCIAL STEAMSHIPS.

Year.	Vessels.	Overall Length.	Displacement Tonnage.
		Feet.	
1889	"Majestic,"	585	10,150
1893	"Campania,"	622	18,000
1898	"Kaiser Wilhelm der Grosse," . .	648	20,880
1899	"Oceanic,"	704	28,500
1906	"Mauretania,"	785	43,000
1911	"Olympic,"	882½	66,000
1913	"Aquitania,"	901	49,400
1914	"Vaterland,"	935	56,000

LENGTH AND TONNAGE OF NOTABLE BATTLESHIPS.

Year.	Vessels.	Overall Length.	Displacement Tonnage.
		Feet.	
1889	"Camperdown,"	330	10,600
1892	"Royal Sovereign,"	380	14,150
1902	"London,"	400	15,000
1906	"Britannia,"	425	16,350
1911	"Orion,"	545	22,500
1912	"Iron Duke,"	620	26,400
1913	"Lion,"	680	30,415
1914	"Queen Elizabeth,"	650	27,500

It is obvious that the area provided for a harbour should not only be adequate for the reception of the largest number of vessels likely to enter, but also that a sufficiency of room should be available for manœuvring them into and out of their berths, and due provision should also be made for alteration in position from natural causes. In the event of a change of wind, and semi-diurnally as regards harbours in tidal waters, vessels riding at single moorings will swing round on their anchorage. Ample space must, therefore, be available to permit of their doing this without the risk of fouling one another, or of colliding with anything in the neighbourhood.

There are several methods of mooring, and the variation in the amount of space required is considerable. A vessel may drop a single anchor, and under this condition the radius of swing will be the vessel's length plus the length of the cable out, which latter is generally three times the depth of water anchored in, and may easily run to 50 or 60 fathoms, even, in certain cases when anchoring in deep water, to as much as 90 or 100 fathoms. The object in having a long lead, much longer than is actually necessary on account of the depth, is to render the pull on the anchor as nearly horizontal as possible, in order that the maximum resistance may be opposed to dragging and the most efficient hold of the ground obtained.

Alternatively, a vessel may be moored by two anchors, in which case she drops the first and then steams ahead the necessary distance—a hundred fathoms, say—to drop the second, afterwards withdrawing to a position midway between the two, where she makes fast. By this arrangement the swing is but little more than that due to the length of the vessel. At the same time, it must be borne in mind that the vessel requires a clear berth fore and aft to drop and raise her anchors, and that the points of anchorage may not, therefore, be encroached upon by other vessels.[1]

Assuming, as an example, a vessel 500 feet long riding at single anchor iwth a length of 50 fathoms of cable, a clear radius of at least 250 yards is required for turning, and this involves an area of about 40 acres. If moored to two anchors, the radius would be reduced to, say, 175 yards, and the area actually swept to between 6 and 7 acres.

It is generally advisable to provide a slightly larger margin for battleships than for merchantmen, owing to the heavier build of the former, and the greater difficulty of handling them in confined areas. It may be taken that at the present time a distance apart of two cables (1,200 feet) is quite adequate for this purpose, and in most cases a cable and a half would suffice.

In sheltered situations, and where the nature of the ground (such as sand or gravel) is not well adapted for ordinary anchorage, buoy moorings may be provided. These will generally be secured to mooring screws sunk by the hydraulic jet or other means into the ground to a sufficient depth (say 10 or

[1] If a vessel moored under these conditions proposes to remain some time, it is advisable to put on a mooring swivel to prevent turns being taken in the cables.

15 feet) to resist withdrawal under stress. Buoy moorings may be used either singly or in pairs. The swing in the former case will depend upon the anchorage of the buoy, whether by single or multiple attachment to the ground. If single, the movement will be less than in an ordinary single anchorage, because, owing to the screw being well embedded in the ground, there is not the same occasion to provide a long lead; if double, the buoy itself will be the pivot of the turning movement. A vessel moored head and stern, of course, does not appreciably change its position, except in the event of accident. If it lies at double moorings, which happen to be at right angles to the direction of the wind,[1] it has its broadside exposed to the full force of the gale, and cases are not infrequent in which a vessel so moored breaks away under the strain from one of the moorings, either through the fracture of a rope, or through the giving way of the mooring cable itself. Such a contingency should be provided for, and the consideration should also not be lost sight of that a small vessel breaking loose through its own defective gear may endanger many of its better equipped neighbours.[2]

The principal object of double moorings is to ensure a vessel's remaining practically stationary during the discharge of cargo overside to lighters and barges. The leads should, however, be sufficiently long to prevent the occurrence of any abrupt jerk on the cable during periods of swell.

Anchorage and Depth.—For the efficiency of its anchorage, a harbour depends upon the nature of the ground and the depth of water obtainable. Light, sandy bottoms do not, as a rule, afford a good hold; but firm, tenacious material of any kind is most suitable. The depths should be sufficient to allow adequate flotation for ships of the largest size. A battleship of the highest class has a draught of about 28 to 30 feet. The draught of the most modern vessels in the mercantile marine now attains to nearly 40 feet, but the greater number draw less than 30 feet of water, and from 6 to 7 fathoms at low water will prove sufficient for present requirements of anchorage, especially in regard to vessels which are likely to seek a harbour for refuge purposes only.

It should be noted, however, that insufficient depth is fatal to the utility and development of any harbour, more particularly in the case of commercial harbours, which have to meet the growing requirements of shipowners and shipbuilders. The dimensions of vessels in the mercantile marine are

[1] This indicates the desirability of arranging head and stern moorings parallel with the direction of the prevailing wind, but in this connection see remarks on Prevailing Wind, p. 26.

[2] "Vessels attach themselves to the bow buoy by means of their own main cables, and to the stern buoy by means of one or more of their own ropes, sometimes wire, sometimes coir or manila, often both. The breakage, owing to the wind of the cables attached to the Trust's bow mooring buoys, is practically unknown, but it is comparatively common for a vessel, especially if one of the minor liners, and not well found, to break her stern line and swing round head to wind."—Spring on Madras Harbour. *Min. Proc. Inst. C.E.*, vol. cxc.

expanding continuously and rapidly, and the greatest difficulty is being experienced in obtaining the proper correspondence in draught owing to the shallowness of the entrances to many harbours and of main waterways, such as the Suez Canal, which is undergoing a process of deepening in successive stages under pressure of necessity. The Panama Canal, a lock canal with masonry sills, is inaugurated with a navigable depth of 40 feet. The Kaiser Wilhelm Canal is being deepened to give 36 feet draught.

Repairing Yards.—Ships once within the shelter of a harbour should certainly find facilities for obtaining supplies of stores and coal, and for executing any repairs which may be necessary to render them seaworthy, or to enable them at least to proceed to some neighbouring port where they may receive proper attention. The extent to which this accommodation is possible, or desirable, will vary with the locality. It is not often that a ship, sufficiently large to require a graving dock for its overhaul, will be driven to seek refuge, but the contingency is possible, and in case of war is not unlikely. Smaller boats may be beached or laid upon a gridiron or slipway.

The circumstances attending an outbreak of hostilities are such as to render it eminently necessary for harbours of refuge to be equipped with adequate means of defence. Under modern conditions of torpedo attack, it has become necessary to protect the entrances of naval stations and commercial ports by special devices. These precautions, however, appertain to the province of military engineering.

Commercial Harbours.—Passing on to special aspects of Commercial Harbours, we may describe them as forming essential features of ports engaged in foreign and coastwise traffic. They constitute the great termini of the highways of the sea. Their province is the accommodation of the mercantile marine during the operations of loading and discharging cargoes, and for the transaction of trade. Thus, in addition to the obviously fundamental needs of accessibility and accommodation already discussed, we meet with the more special requirements of Quays and Sheds, and also of Inner Basins and Repairing Docks.

Commercial harbours are to be found in a variety of situations: upon the seacoast, at the mouths of rivers, inside sheltered estuaries, and even some considerable distance inland along the banks of rivers and canals. They require more shelter than that which suffices for simple purposes of refuge. It is indispensable to the conditions of modern trade that there should be the least possible delay in the reception and despatch of vessels; hence everything must be done to ensure continuity of operations, and for this purpose protected quays are a first consideration.

Coastal Harbours present most difficulty in regard to this point. The mere protection afforded by a breakwater is not sufficient to impart that tranquillity which is essential to the loading and unloading of ships. There are, of course, cases like that of Zeebrugge (fig. 4) where a single mole built

out into the sea is made to suffice, being provided with a level quay and covered sheds for the reception of merchandise; but such cases are rare, and, generally speaking, it will be found necessary to provide an enclosure, practically complete, with an inner harbour, or docks, for commercial purposes. Indeed, in the case of a coast exposed to heavy seas, the adjunct becomes imperative.

Estuarine Harbours find the requisite shelter already provided, in many cases, by rising ground flanking their entrances, and, indeed, many harbours situated in creeks and inlets are also admirably protected by adjacent hills so as to require no further defence. In addition to Cork and Queenstown, the

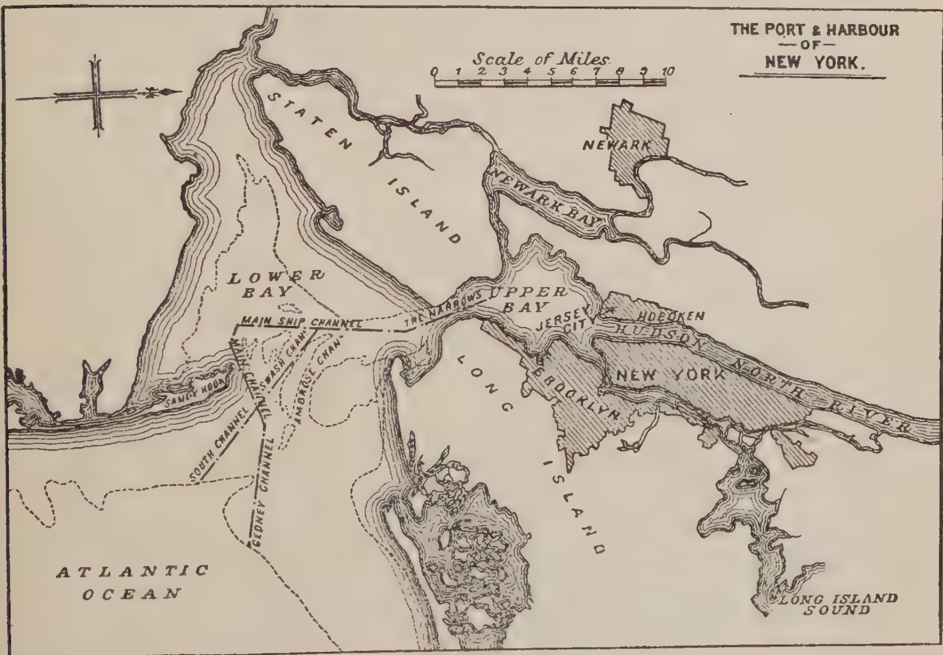


Fig. 10.—New York Harbour.

Note.—The Ambrose Channel, with a dredged depth of 40 feet, is now the principal approach to the port for shipping.

harbours of Sydney, San Francisco, and Rio de Janeiro may be cited as cases in point. Instances of estuarine harbours are afforded by Liverpool, at the mouth of the Mersey; Dublin, at the mouth of the Liffey; Havre, at the mouth of the Seine, and numerous other ports. The case of Liverpool is an admirable example, the form of the river at its mouth being excellently adapted for harbourage. The Mersey is broad and deep, expanding inwards from a narrow-necked entrance, while rising ground on both sides contributes the necessary shelter.

But while estuarine harbours possess many advantages, they have corresponding defects. The majority of rivers are afflicted with bars—that is to

say, ridges of material lying across their entrance in such a manner as to reduce the available depth of water, and so impede navigation. The point is only mentioned here in passing: it is of such grave importance as to call for detailed treatment, and this must be reserved for a later chapter.

River Ports, such as Bremen on the Weser, Hamburg on the Elbe, Glasgow on the Clyde, etc., and Canal Ports, such as Manchester, Bruges, etc., are comparatively free from many of the evils which affect harbours on the littoral, but they are attended by certain inconveniences of another kind. The navigation of inland waterways is a slow and tedious process for sea-going ships, and it involves considerable delay, which, in these days of rapid transit, counts for a great deal. Ports like Antwerp, Hamburg, and Rotterdam are very unfavourably situated for competing, as regards expedition and despatch, with ports on the seacoast. True, there are advantages attaching to water carriage throughout, which outweigh the alternative of unshipment and transport by rail from the nearest seaport; but this fact does not invalidate the contention that the nearer the port to the great ocean highways, the speedier and better the distribution of freight. Moreover, river harbours are subject to the adverse influences of strong currents, freshets, and floods; they have a continuous tendency to silt up, and they are even invaded at times by floating masses of weeds and mud. Under these particular circumstances, a river harbour should possess an entrance pointing downstream, making an acute angle with the bank, and the width of entrance should not be greater than is necessary for the admission of the vessels which frequent the port.

Fishery Harbours.—Fishery Harbours, though a numerous class, are not, generally speaking, constructionally important, but they possess characteristics sufficiently notable to merit some attention. In the first place, the scarcity of capital available for artificial operations renders it necessary to take the utmost advantage of natural features. Then, since fishermen require a maximum amount of time for their expeditions, with a minimum of delay in despatching their hauls, on account of the perishable nature of fish, every facility should be afforded them for making the harbour at the last possible moment consistent with the state of the tide. This intermittent accessibility, however, characteristic as it is of all tidal harbours in which vessels take the ground at low water, or where there is a shallow bar, is inimical to the interests of sea fishermen. Their requirements demand a harbour constantly open for arrival and departure alike. A shoal of fish may be missed as easily from sheer inability to proceed to sea as from the deterrent effect of impending foul weather.[1]

[1] Depths below low-water level of ordinary spring tides at some of the more important fishery harbours in this country are:—Hartlepool, 15 feet; Aberdeen, 14 feet; Dundee, Yarmouth, and Newlyn, 12 feet; Lowestoft, 10 feet. The majority of fishery harbours, however, have depths much less than these, and, in many cases, they become practically or actually dry at low water.

The **entrance** of a fishery harbour, while not unduly wide, must not be made too narrow. It is liable to be thronged at times by craft anxious to enter in order to escape a rising gale, or to catch the early market. Cases have occurred in which boats have become jammed in an entrance through excessive crowding, with, unfortunately, disastrous consequences. Fishing smacks of the present day have a beam of 20 feet or so, and allowance should be made for, at least, three or four, or even more, entering abreast, according to the size of the fleet. This means that 100 feet should be regarded as the minimum width entertainable. It is true that some small harbours are, or have to be, content with less than this, but they are exceptional cases controlled by their special and restricted environment. The majority of

Fig. 11.—Entrance to Langston Harbour. Direction of currents. The numerals represent hours of ebb and flood tide.

minor coastal and fishery harbours possess entrances between 100 and 300 feet in width. The entrance width of Ramsgate harbour is 200 feet; of Banff harbour, 150 feet; of Fraserburgh harbour, 180 feet; of Lowestoft harbour, 150 feet; of Grimsby harbour and of Yarmouth harbour, 250 feet. Fowey has an exceptionally wide entrance of 780 feet, and Boscastle an exceptionally narrow one of 60 feet.

Size.—It has already been indicated that the majority of fishery harbours have areas of less than 10 acres, but in certain cases, as at Grimsby and Fowey, the area may reach as much as 100 acres. Looe has a harbour of

60 acres, Fraserburgh a harbour of 20 acres, Lowestoft of 24 acres, and there are some others of similar areas.

Much depends upon the size and number of vessels to be accommodated, provided, of course, the locality itself imposes no over-riding restriction. The length of ordinary fishing boats ranges from 50 to 60 feet, but steam trawlers are rarely less than 110 or 120 feet long. These constitute the class called "Fleeters," because they fish in fleets. "Single boaters" fish separately, making sea excursions of a week or so's duration, and as a class are somewhat larger than Fleeters. "Icelanders," which go further afield, are still larger vessels capable of remaining at sea for three or four weeks at a time.

Depth.—The loaded draught of the smaller craft lies generally between 6 feet and 10 feet, but steam trawlers draw as much as 15 to 18 feet, or even more.

Equipment.—In a fishery harbour, broad open quays with a large covered hall, or market, are required for sorting the fish and conducting sales.

It has to be borne in mind that under modern conditions of trade, a fishing fleet of steam trawlers must needs be furnished with supplies of coal and ice, and for this purpose railway sidings at the quayside become a necessity. For large vessels of this class, the fishery harbour becomes of a more important character, trenching, in many respects, on the commercial harbour.

Selection of Locality.—Reviewing the subject as a whole, and postulating the choice of a situation for a harbour, the conditions which would govern that choice divide themselves into three heads.

First and foremost, there is the obvious advantage to be derived from a position adjacent to some existing means of internal communication, such as river, canal, or railway. In the absence of all these, it is still possible to consider the feasibility of the formation of two of them, and any obstacles likely to prove antagonistic should be carefully weighed and avoided.

Then the extent of adverse meteorological and climatic influences claims attention. Fog and frost are dual evils which infest very many harbours to the infinite detriment of their usefulness. The former jeopardises shipping, and both impede navigation. To be ice-locked for several months in a year, like Montreal or Petrograd, is a serious outlook for any progressive port. The ice-breaker, however, has done much to relax the rigorous grasp of winter, and very few ports need now resign themselves to the entire cessation of their sea-trading operations. For the fog-fiend, there is unfortunately no practicable remedy as yet. Certain experiments conducted in scientific quarters seem to suggest a possible amelioration, but on too small a scale for general application. As regards protection from storms, the value of headlands and promontories has already been pointed out. Cover should, of course, be sought from the most tempestuous quarter.

Lastly, the facilities afforded for providing such artificial protection as is requisite should be taken into account. This constitutes the economical aspect of the question, and some of the numerous matters deserving attention are the length of breakwaters and jetties required, the nature of their construction, the source of suitable material, and the expense of carriage, the means of procuring labour and plant, and the resources of the district generally. In regard to all these, certain localities will be more favourably situated than others.

But, except in rare instances, the engineer can hardly expect to have the opportunity of allocating a harbour and of designing it *in toto*. Trade routes are sufficiently firmly established to preclude the diversion of much traffic to other lines. An occasional harbour of refuge, with a fishery station or so, marks the limits of entirely new construction at the present day. Yet, at the same time, there is great and increasing scope for the development of maritime works already in existence, and the enlargement and improvement of harbours forms one of the most important fields of civil engineering.

Procedure in Design.—Such being the case, the unrestricted choice of a site will rarely lie within the province of the engineer. The locality, at any rate, will already have been determined and the preliminary dispositions established, before his services are requisitioned. It falls to his lot, therefore, to utilise existing conditions and to devise a *modus vivendi* out of circumstances beyond his control.

Assuming, momentarily, for the purpose of discussing the question in all its bearings, that the site is a virgin one, there are certain preliminaries to be carried out before any scheme can be laid down. We will deal with them in their natural order of procedure. Thus, the first point would be to make a survey of the neighbourhood, and to prepare a chart indicating the depths of water in the vicinity. Not only should a complete set of soundings be taken, but borings should also be made to ascertain the nature of the ground, its fitness for anchorage, and the extent to which it lends itself to an economical increase of depth, should this be or become necessary. The depths obviously must be sufficient to meet the requirements of the deepest draughted vessels which are likely to frequent the place, and it should not be overlooked that some allowance is necessary for the pitch or surge or "scend" of a vessel in rough water, whereby its keel descends below the normal level.[1]

Natural Phenomena.—After the preparation of the survey and the plotting of the contour lines (or lines of uniform level, as shown in fig. 4), the engineer

[1] In certain districts subject to heavy swell the margin required may be considerable. It was established by a Court of Marine Inquiry at Melbourne that the s.s. "Cufic," drawing 28 feet, injured her hull and propeller through touching ground at the entrance of Port Philip, while encountering some exceptionally heavy rollers in a depth of 39 feet of water. *Min. Proc. Inst. C.E.*, vol. cxc., p. 181.

will search local records for data, and also make observations himself, in reference to various natural and metereological phenomena, and the following will specially claim his attention :—

1. The direction and intensity of the winds and the frequency of storms.
2. The height and force of the waves.
3. The range of the tides.
4. The direction and velocity of the currents.
5. Evidences of silting, littoral drift, or coast erosion.
6. The extent of exposure and the maximum "fetch."

With reference to the first of these features, it may be pointed out that nearly every place is subject to what is called a **Prevailing Wind,** that is, a wind blowing with great constancy for a considerable portion of the year from a certain point of the compass. But while the prevailing wind is the most frequent, it must not, by any means, be concluded that it is the wind which is most to be feared. A single gale arising from a totally different quarter may cause more havoc and destruction than a whole twelve-month of the prevailing wind. The importance of the latter lies rather in the effect it has upon the coastal contour in its relationship with other agencies, the effects of which, though momentarily insignificant, are continuous and cumulative. Such agencies are the ebb and flow of the tide and the erosive and transportive power of waves and currents.

Methods of recording wind frequency and intensity are numerous. Three examples are shown in figs. 12, 13, and 14, with explanatory notes. Time ordinates are not difficult to plot, possessing, as they do, a direct numerical value. It is a different matter with the intensity ordinates, which have to be to a certain extent conjectured. The velocity of a wind may be gauged more or less accurately by an anemometer, if one is available; in other cases it is estimated. But the velocity is fitful, and as a measure of pressure, by no means an ideal standard. The range of intensity is usually divided into twelve sections, forming a scale, known as Beaufort's scale, which is given below.

Beaufort Scale for Wind.

0	denotes	Calm	Velocity in miles per hour		= 0
1	,,	Light Air	,,	,,	7
2	,,	Light Breeze	,,	,,	14
3	,,	Gentle Breeze	,,	,,	21
4	,,	Moderate Breeze	,,	,,	28
5	,,	Fresh Breeze	,,	,,	35
6	,,	Strong Breeze	,,	,,	42
7	,,	Moderate Gale	,,	,,	49
8	,,	Fresh Gale	,,	,,	56
9	,,	Strong Gale	,,	,,	63
10	,,	Whole Gale	,,	,,	70
11	,,	Storm	,,	,,	77
12	,,	Hurricane	,,	,,	84

Coastal Change.—The influence of the wind in relation to the magnitude of waves will be more fully considered in another chapter. At present, having regard to the general outlines of harbour design, we will simply notice its bearing upon the coastal contour in the vicinity of any artificial works.

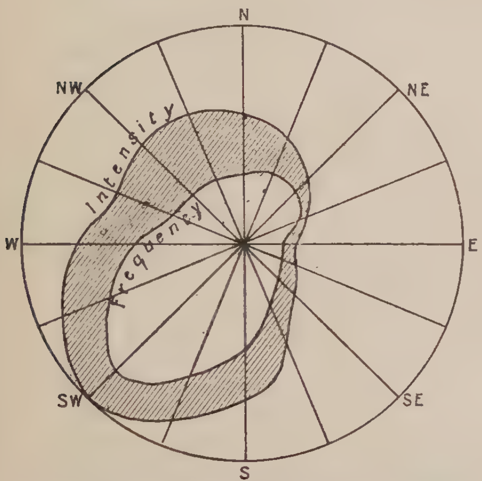


Fig. 12.—Wind Diagram. Frequency ordinates set off from centre; intensity ordinates from frequency curve.

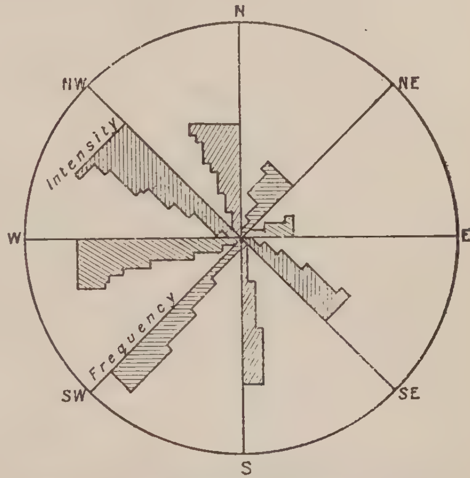


Fig. 13.—Wind Diagram. Frequency ordinates set off from centre; intensity ordinates from radial lines.

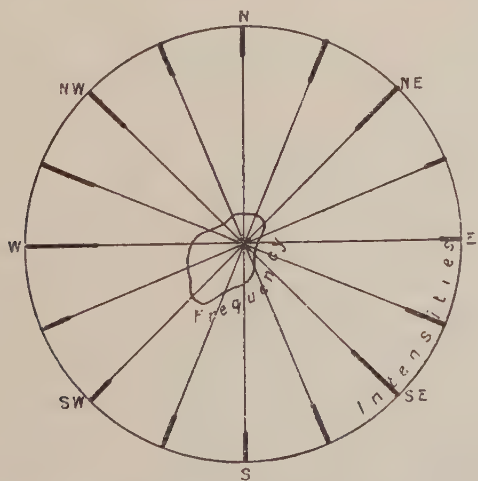


Fig. 14.—Wind Diagram. Frequency ordinates set off from centre; maximum intensity ordinates from circumference.

That the seacoast is undergoing a gradual change must be evident to the most superficial observer. In certain districts, notably the borders of Yorkshire and East Anglia, there are manifest signs of sea encroachment. Every

year witnesses the retrogression of some extent of shore frontage, and, in the course of a few centuries, whole tracts, such as the Goodwin Sands, and districts, including villages and townships, have disappeared. On the other hand, in other quarters there has been a gradual gain and accretion. Southport in Lancashire, formerly, as its name implies, situated at the water's edge, now lies at an appreciable distance inland. At Dungeness in Kent, a headland of shingle is accumulating at something like the rate of 200,000 tons per annum. Instances of both kinds might be multiplied indefinitely.

The essential point to consider is the probable effect of any artificial projection from the coast in accentuating or mitigating the natural process of mutation. This is not altogether an easy matter to determine, owing to the predominating influence of local circumstances, quite apart from the fact that the causes of coastal denudation and accretion are but imperfectly understood. The carriage of material from one point to another is assigned by one school of engineers entirely to wave-action, and by another school, mainly to current flow.[1] The evidence, on the whole, renders it most probable

[1] The following may be cited as instances of this divergency of opinion :—

"Travel of beach is, on the whole, governed by the main tidal currents. The flood stream where it impinges on a coast line, carries the beach with it in its course."—W. T. Douglass on Foreshore Protection, Engineering Conference, 1903.

". . . The one great principle, which is the key of the whole question, . . . is that the force by which the sand (at Madras) is disturbed and transported is that of the waves and not of the currents. . . ."—W. Parkes, quoted by Sir F. Spring in Coastal Sand Travel near Madras Harbour, *Min. Proc. Inst. C.E.*, vol. cxciv.

"The principal factors which govern the movement of sand and shingle on the littoral are: (1) ocean and tidal currents; (2) wave action; and (3) wind. On any coast where the tidal range is large, say from 12 to 20 or more feet, the flood tide is the predominant influence and its direction indicates the course which sand and shingle will take on the beach."—G. H. Halligan, Sand Movement on the New South Wales Coast, *Proc. Linnean Soc. of N. S. Wales*, vol. xxxi.

"Where waters are shallow, wind-waves in heavy gales plough up the sandy bed along a coast. This sand is kept more or less in suspension, is borne by the littoral currents, and is subsequently deposited over wide areas. . . . In deep water, under normal conditions of current, a totally different effect is produced. The current then traverses the surface of a sandbank, following the irregularities of its level, and causing but little disturbance."—A. E. Carey on the Sanding up of Tidal Harbours, *Min. Proc. Inst. C.E.*, vol. clvi.

"While wind and waves are the agents which operate in eroding the cliffs and producing the drift, the regular and continuous travel of the material along the coast is due to the wave-action of the flood-tide."—W. H. Wheeler on Littoral Drift, *Min. Proc. Inst. C.E.*, vol. cxxv.

"Much of the testimony concerning littoral drift and the dynamic action of the flood-tide had been collected by such men as wreckers, light-keepers, and surfmen, whose observations of facts he believed to be reliable, and forced him to the conclusion that such movements did exist in a direction opposed to prevailing winds, and that the motor was the angular movement of the waves and 'breakers,' especially during flood-tide."—Lewis M. Haupt in Discussion on Mr. Wheeler's Paper on Littoral Drift. *Vide* also Haupt on Littoral Movement on the New Jersey Coast, *Trans. Am. Soc. C.E.*, vol. xxiii.

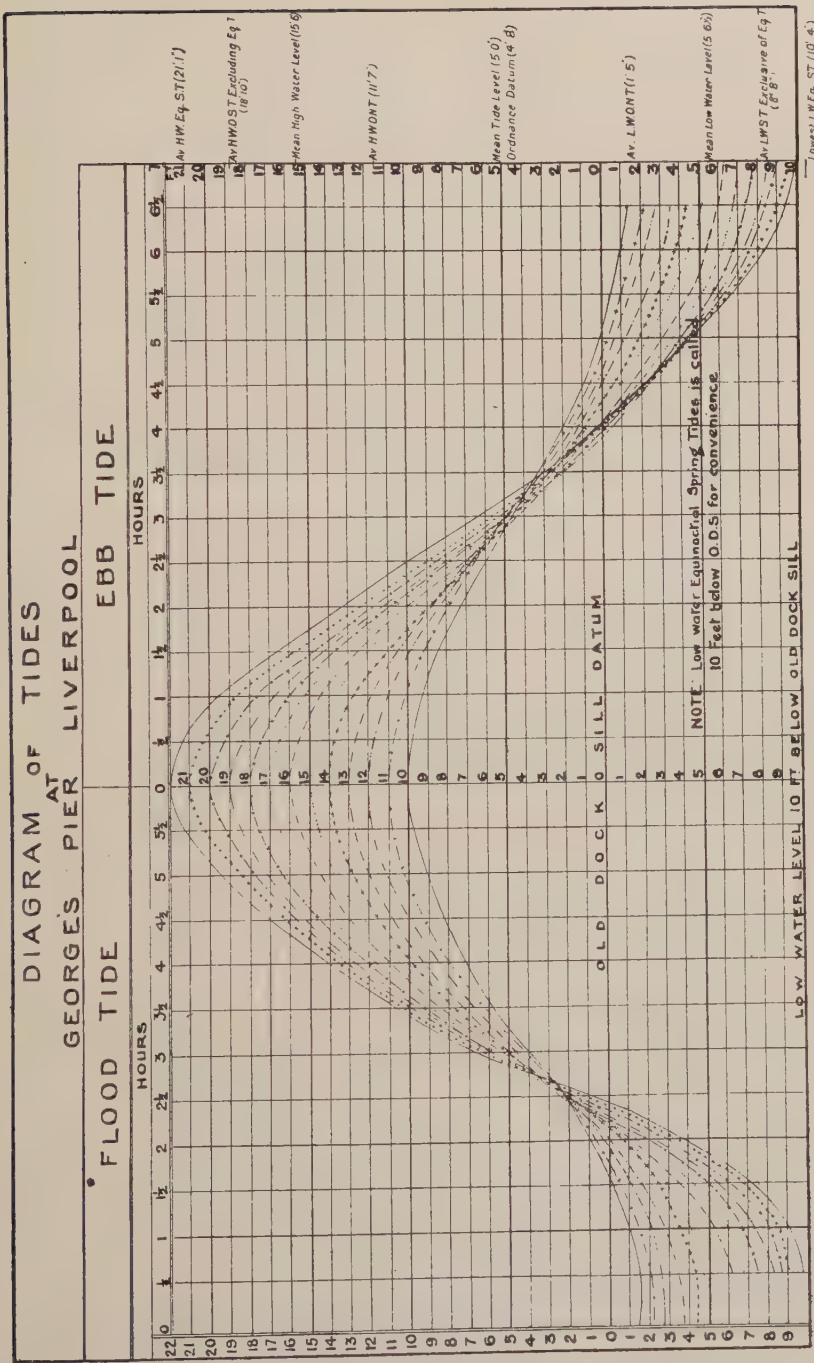


Fig. 15.—Tidal Diagram, showing range of tides at Liverpool.

that both agencies are involved, in varying degree: the breaking of waves on a beach serves to stir up the sand and shingle, the former of which the water, in its troubled state, retains in suspension long enough for it to be projected some distance along the shore by the resolved component of wave force in that direction, the movement being assisted, more particularly as

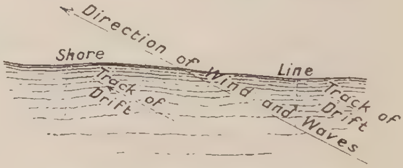


Fig. 16.—Track of Shore Drift.

regards the lighter material, by the littoral current. The heavier particles are rolled along the bottom and partake of a zig-zag movement, as shown in fig. 16. It is generally agreed that the action is practically confined to the region between high and low water marks.

The trend of littoral drift is, therefore, attributable, in the first instance, to the wind which governs the predominant direction of the waves.

To illustrate in some way, however imperfectly, the general effect of wind and flow upon a coast-line, with the modifications brought about by intrusive structures, figs. 18, 19, and 20 have been prepared. A simple case only has

Fig. 17.—Salina Cruz Harbour. Effect of initiatory breakwater operations.
Note.—The design was subsequently modified.

been taken; the action, as can well be imagined, is often much more complex. The supposition made is that of a coast-line with the dominant current flowing parallel thereto (from right to left), and with the prevailing wind

making an oblique angle so as to give a component in the same direction. Fig. 18 and those which follow serve to indicate the tendency towards shoaling in various parts, which is brought about by the construction of harbour works of typical kinds.

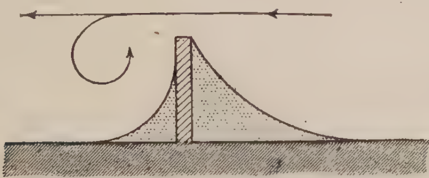


Fig. 18.

The straight pier or breakwater at right angles to the coast-line (fig. 18) induces an accretion of sand and shingle along each of its sides. The windward accumulation is the more pronounced, the leeward deposit being reduced by eddying round the outer end of the pier.

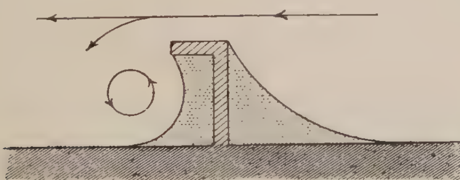


Fig. 19.

The returned pier (fig. 19) serves to increase the leeward deposit, there being a circular motion of the water round the pierhead with a tendency to scour at that point, while the slacker water inside leads to settlement of suspended material. An example of this is to

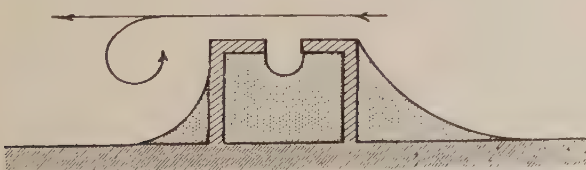


Fig. 20.

be found in the harbour of Salina Cruz on the Pacific Coast (fig. 17), where the initiation of a breakwater of this type brought the low-water line forward, temporarily, at any rate, to the 26 feet contour of six years

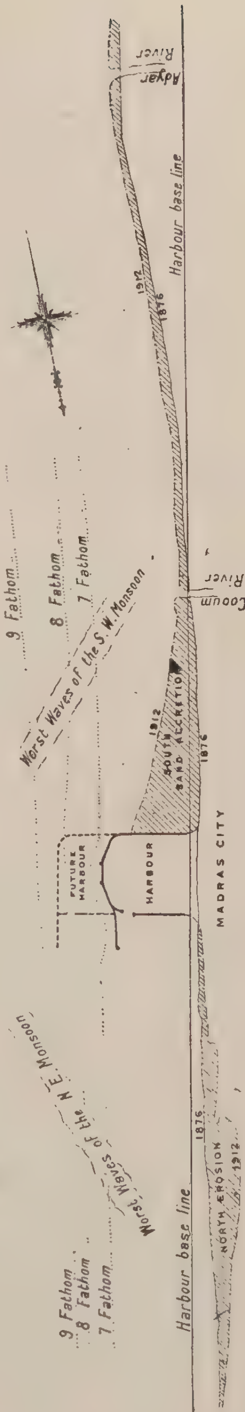


Fig. 21.—Sand Accretion and Coast Erosion at Madras.

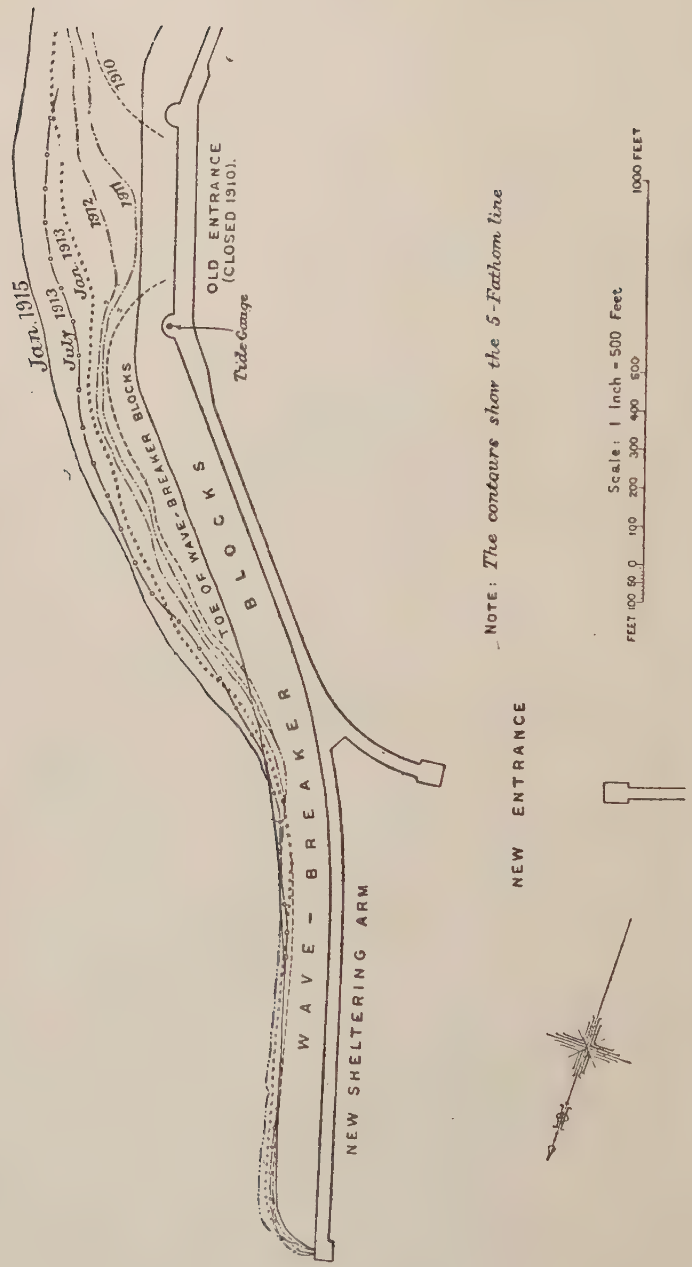


Fig. 22.—Madras Harbour: Plan showing the advance of the Sand from the South in the years 1910-1915.

previously. Much the same effect is apparent with double piers (fig. 20), the accretion being emphasised by reason of the additional extent of quiescent area. Confirmatory evidence is forthcoming from Madras, where the earlier harbour entrance was slowly but surely silted up, and for this and other reasons a new entrance had to be provided in a different position.

The case of Madras is, indeed, particularly instructive, and merits some detailed notice. The harbour breakwaters extend outwards about 3,000 feet from the original low-water line, as it was at the commencement of the work of construction in 1876. Up to the year 1913 a large triangular area of sand, about 260 acres in extent, had accumulated on the southern side of the projection, with a base of 9,000 feet along the coast line, and a side of 2,000 feet along the breakwater.[1] Moreover, as shown in fig. 21, this was not the whole of the accretion, for a narrow strip parallel to the shore had also been formed, extending from the end of the triangle for a distance of nearly 4 miles to the mouth of the River Adyar, and enclosing an area of 114 acres. On the other hand, northward of the harbour, there had been a corresponding erosion for a length of 3 miles along the shore, and this it had been found necessary to check by means of stone revetments.

The old entrance to the harbour was centrally situated between the breakwaters, facing east, and the sand drifting northwards found slack water between the pier heads wherein to settle, with the result that, before it was closed, the entrance was shallowing up at the rate of just 1 foot per annum over a period of twelve years. With the closing of the old entrance and the extension of the eastern arm, the deposit has continued, and there is uninterrupted accretion along the whole eastern face (fig. 22) which promises to become more and more pronounced unless checked by artificial measures.

Openwork and Island Harbours.—Apparently the only practicable means of remedying the evil due to solid structures is that of substituting openwork for the portion of the jetty which immediately joins the land. It has even been suggested that the most logical method is that of "Island Harbours," formed in deep water out of the range of accretion, and connected with the shore by means of openwork jetties, as exemplified at Arnager, Snogebœk, and Hundested (p. 54). Columnar structures offer little, if any, perceptible obstruction to current flow, and consequently should not give rise to a deposit of any appreciable extent. Whatever shoaling takes place abreast of the outermost and solid portion of the pier should be, and generally is, comparatively insignificant. The breakwater at Zeebrugge has been designed on these lines, as also the pier at Rosslare on the east coast of Ireland.

Cearà Harbour.—Unfortunately, for the general application of the principle, a disposition of the same kind adopted at Cearà harbour, in Brazil, has proved strikingly unsuccessful, and has entailed consequences disastrous

[1] *Vide* Spring on Coastal Sand Travel near Madras Harbour, *Min. Proc. Inst. C.E.*, vol. cxciv.

to the port. The entire harbour is blocked with sand (fig. 23), despite the fact that an openwork viaduct lies between the breakwater and the shore.

The want of success is, no doubt, attributable in some degree to the closeness of the piling in the viaduct. There are four rows of piles 8 feet 9 inches apart, with 30-foot spans. It is said that, as fast as the piles were driven, a tongue of sand followed them out from the shore, so that the 750 feet opening provided for was never actually realised.[1]

The design of Cearà harbour has evidently been based on the analogy of the Coroa Grande, or Great Reef, which fringes the coast, and largely confines the littoral current within the channel thus formed. But while the top of the breakwater is 16 feet above low water of ordinary spring tides, the top of the reef is some 10 feet below the same datum, and the reef, therefore, does not prevent a certain amount of wave action sufficient to disturb the

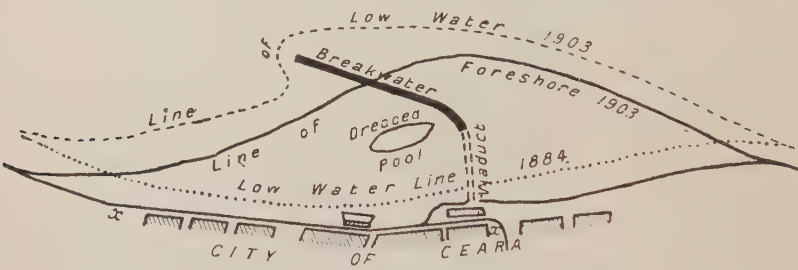


Fig. 23.—Harbour of Cearà. Effect of breakwater.

sand and keep it from settling permanently. The absence of this agitation in the case of the area within the breakwater is largely responsible for the silting up which has taken place, though, if the current could have been maintained in undiminished force through the viaduct, the harmful results would probably not have been so pronounced. The current at Cearà is, however, of no great intensity, being only $\frac{2}{5}$ knot on flood tide and $\frac{3}{10}$ knot at ebb, and the range of tide is only about $8\frac{1}{2}$ feet. The coast, moreover, is bordered by great sand dunes, which yield an unlimited supply of sand when subjected either to wind or wave action.

The author has been much interested in the problem, and when certain data were placed before him and his opinion invited in 1910, he expressed the view that the only artificial work which would have any chance of avoiding the shoaling difficulty would be a curved breakwater following the line of the reef, with a trumpet-shaped expansion at each end, so as to concentrate and intensify the littoral flow. Such a breakwater would give a certain amount of protection to shipping, but the environment was unfavourable to the formation of a thoroughly sheltered harbour. If shore connections were constructed, it would be necessary to maintain dredging operations on an extensive, though not necessarily prohibitive, scale. Some scheme of

[1] *Min. Proc. Inst. C.E.*, vol. clvi., p. 269.

this nature has independently been recommended for adoption by Brazilian engineers, but no active steps have yet been taken in the matter.

For information respecting the projected design, the author is much indebted to Senhor Manoel Bandeira and to Dr. F. Burlamaqui. The former, writing in 1909, made the following observations :—

"Comparing Sir John Hawkshaw's plan with ours, it is clear that there is a current strong enough to maintain a channel between the littoral and Coroa Grand Reef, and notwithstanding the large deposit of sand caused by the construction of the breakwater, that channel was conserved at the same depth —even a little deeper on the west side—and only became a little narrower. The observation of currents made by us is consistent with the general result, because we found a velocity almost always greater than 0·15 metre (6 inches) per second, and, at that speed, a current is able to convey fine sand, like the dune sand of Cearà. The set of currents is always from east to west, at whatever season or state of the tide. Winds there have practically always the same direction, S.S.E. to S.E.; and only at the rainy seasons veer to the north. Wave direction is always more or less N.E."

Effect of River Flow.—Land water entering the sea at any point is deflected by tidal currents, where they exist, to each side alternatively, with the result that with the coastal sediment there is a tendency to shoal at some short distance outwards, forming a bar of various contours. With a strong wind and littoral current in one predominant direction, there will most likely be produced, in addition to a bar, a spit or horn (fig. 24), through which the river, in times of flood, may break, but which generally reforms. In either

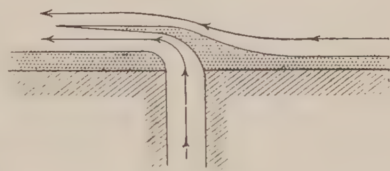


Fig. 24.

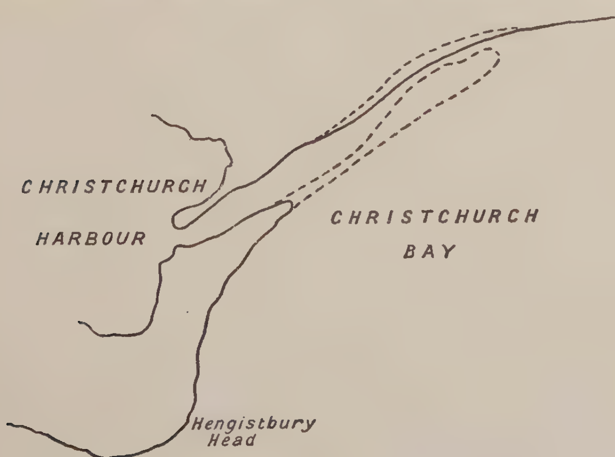


Fig. 25.—Entrance to Christchurch Harbour. Formation of spit.

case the shoaling is detrimental, and various means have been tried to remove it. The most generally accepted methods are by means of projecting jetties, or training walls, and dredging. Unless the former are extended into very deep water, they are not likely of themselves to prove completely efficacious. Accretion takes place as shown in fig. 26, and matters readjust themselves a little further out much to the same effect as before. The history of Dunkirk, Calais, and other French ports for a period of over two centuries is a continuous record of jetty extension from time to time, as shoaling has accrued. The works at the mouth of the Mississippi (where there is, of course, some difference in the tidal conditions)[1] are of the same nature, being designed for the removal of a bar; but the jetties are of considerable length, and, being extended to a point sufficiently seaward to bring the fluvial deposits within range of powerful submarine currents, there is less fear of the deposits becoming permanently localised to any serious extent. At the same time, the jetties may need extending ultimately on account of the advancement of the coast-line.

A theory has lately been advanced that the narrowing of a tidal inlet only serves to increase the internal silting, instead of diminishing it, as intended. While this theory does not meet with general acceptance, and, indeed, has encountered much criticism, it is of interest as indicating an original point of view, and it may best be stated in the exact words of its propounder, Mr. G. H. Halligan, F.G.S., Hydrographic Officer to the New South Wales Government. He says:—

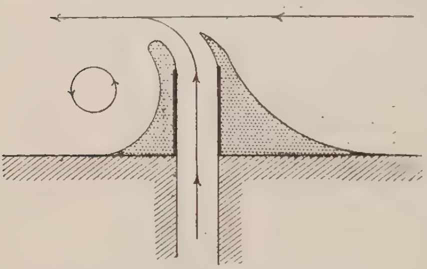


Fig. 26.

"When the sand, put into suspension by the waves, and slowly moving with the current along the beach, at length reaches an inlet, it is carried into the estuary by the flood-tide. At slack water this sand is deposited within the estuary, and as there are no waves to agitate the sand within the estuary, the ebb current, which is only just equal to the flood stream, is unable to remove the whole of the sand which has been brought in. An accumulation thus takes place, and is the cause of all the trouble to the harbour engineer.

"It is quite clear that the narrower the entrance, the stronger the flood-tide will be; and the stronger the flood-tide the more sand will be carried into the estuary. The velocity of the flood-tide can be increased only by contracting the entrance, and decreased only by widening the entrance, and it, therefore, behoves the harbour engineer to keep the entrance as wide as the circumstances will permit. If possible, the width should be so determined that the velocity of the ocean current flowing past shall be greater than the velocity of the flood-tide between the ends of the breakwaters.

[1] The tidal range in the Gulf of Mexico is very small.

The sand will then be carried past in the stronger stream, and will be harmless.

"It will be seen that at Sydney there is 1 foot width of entrance to every 2·81 acres of water, at Botany Bay 1 foot to 4·11 acres, and at Jervis Bay 1 foot to 2·44 acres. With the exception of Jervis Bay, none of these entrances is so wide as to allow dangerous seas to run far from the entrance, so that, if the author's contention is correct, when designing future harbours, a ratio of 1 foot width of entrance to 3 acres of tidal compartment may safely be allowed; but unless it is intended to saddle posterity with heavy bills for dredging, it would appear that the ratio should not exceed 1 to 4." [1]

There are several statements in Mr. Halligan's exposition which are decidedly of a contentious nature, and one of the most important—viz., that the ebb current is only just equal to the flood stream—is certainly not applicable in any general way, whatever may be the case at the harbours where Mr. Halligan's observations were made. The scouring action of the ebb-tide has undoubtedly proved in many instances a powerful agency for conveying sedimentary deposits out to sea, and training works concentrating this action within definite limits have been attended by beneficial effects which are beyond dispute.

With dredging employed as an auxiliary, the silting up of outlet channels may, of course, be held in check as long as and to whatever extent may be deemed expedient. We revert to this part of the subject later on (Chapter X.).

In considering the diagrams employed in illustration of the foregoing principles and hypotheses, it is to be distinctly understood that the shoaling indicated does not necessarily appear above water level, even at lowest water, nor indeed does it, in a number of cases, manifest itself to any pronounced extent. Certain tendencies only are outlined, which become more or less marked, according to the absence or presence of counteracting influences.[2] It is the province of the engineer to secure or provide these counteracting influences by natural or artificial means, so as to maintain a state of equilibrium in so far as it is possible to do so. Some of the means employed for the purpose will be discussed in a later chapter, but we cannot hope to deal exhaustively with a problem which admits of innumerable variations in accordance with local peculiarities. The subject of littoral drift and its complementary phenomenon, coast erosion, is so involved, and its ramifications so extensive, as to require special and individual treatment.

[1] Halligan on Harbours of New South Wales, *Min. Proc. Inst. C.E.*, vol. clxxxiv.

[2] An interesting set of experiments on a model coast-line, with typical projections formed of plasticene, in a lead-lined tray covered with sea sand, in which tidal and wave action were artificially produced approximately to scale, is described in *Engineering* of 19th and 26th September, 1913. The experiments formed the subject of a paper contributed to the British Association Meeting of that year by Prof. E. R. Matthews, who carried them out. The results were in complete conformity with the principles enunciated in this chapter.

As a forcible illustration of the complex nature of currents set up by artificial works, a series of illustrations (figs. 27 to 32) are given, showing the vagaries in the tidal flow exhibited at Dover Harbour since the formation of the new breakwaters. Beforetime, the set of the tide was almost parallel to the shore line, running from N.E. to S.W. from $4\frac{1}{2}$ hours after to $1\frac{3}{4}$ hours before high water, and from S.W. to N.E. from 2 hours before to 4 hours after high water. At high water of spring tides the rate of the east-going stream was about 4 knots, and at low water the west-going stream had a velocity of $2\frac{1}{2}$ knots. Both direction and rate of flow are now completely altered in every respect.[1]

The currents at the entrance to Langston Harbour are indicated in **fig. 11.**

Harbour Areas and Entrance Widths.—We turn now to the subject of wave power, regarding it simply as it affects the general question of harbour design. At a later stage it will be necessary to take it into detailed consideration from the point of view of its influence on structural features. For the moment, however, we are only concerned with it in its general aspect —that is, in so far as it affects the important relationship existing between the area of a harbour and the width of its entrance.

The determination of area is the primary consideration. Obviously, small harbours will be more appreciably affected by external commotion than large harbours, assuming the inclosed areas in each case to be equally well protected, for it is easier to transmit agitation to a small body of water than to a large one. But, on the other hand, large harbours, unless most effectively screened and sheltered, are themselves liable to act in some degree as wave generating areas. Hence some discrimination is necessary, and the question of area is more likely to be determined by other considerations than those immediately connected with exposure. The required accommodation, the dictates of convenience to navigation, and the adaptability of natural features, in fact, have foremost place in the determination of area.

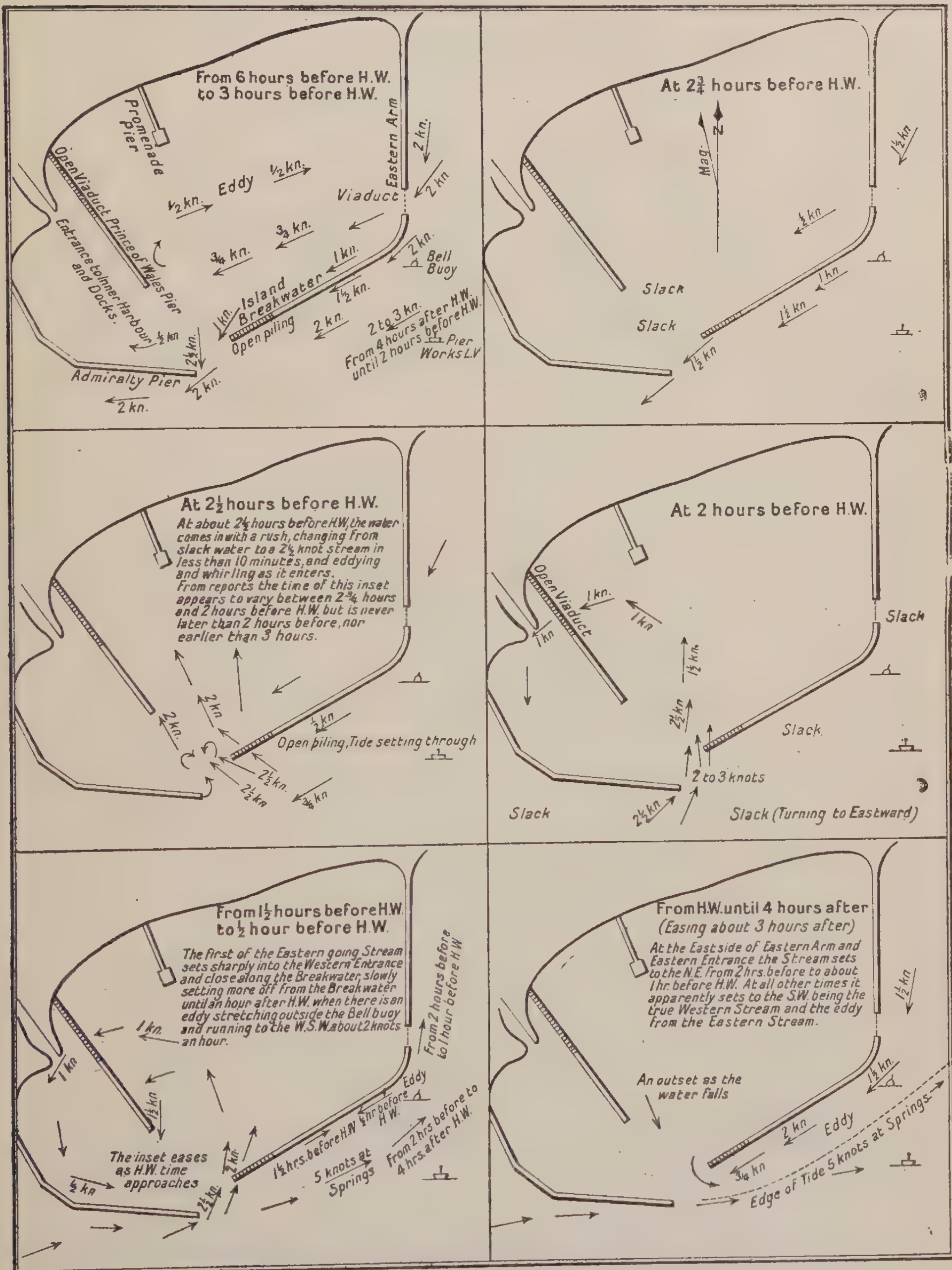
As regards entrance width, there can be little doubt that the narrower the entrance, the more effectually is the interior secured from the ingress of disturbing elements. Moreover, a narrow inlet very materially reduces the power of those waves which do find an entrance. On the other hand, an entrance must have adequate width for vessels entering not only singly and in calm water, but also when driven in groups under stress of weather. Accordingly, the entrance bears a double relationship to the harbour—viz., (*a*) as regards shelter, and (*b*) as regards accommodation.

From the first of these points of view, Stevenson has evolved an empirical formula to connect the extent of the reduction in the height of waves with the width of the inlet and the width of the sheltered area.

[1] When these diagrams (issued by the Admiralty) were prepared, the western portion of the Island breakwater was still incomplete and the site occupied by open constructional piling. The completion of the solid work would lead to some slight modification of the currents at this point, but the illustration as it stands is sufficiently instructive.

H.W. F. & C XI.h 12m
Springs rise 18 feet 9 in. Neap rise 15 feet.

Note: *The height of tide and strength of streams are liable to considerable alteration by strong winds in the North Sea or English Channel.*



Figs. 27 to 32.—Tidal Currents in and out of Dover Harbour.

Thus, calling H the height of the unrestricted wave at the mouth of the harbour having an entrance width b, the reduced height of the wave, h, within the harbours at a distance, D, from its mouth and at a point where the breadth of the harbour is B, is given by the expression

$$h = \mathrm{H}\left\{\sqrt{\frac{b}{\mathrm{B}}} - \cdot 02\sqrt[4]{\mathrm{D}}\left(1 + \sqrt{\frac{b}{\mathrm{B}}}\right)\right\}.$$

For example, when D = 256 feet, b 100 feet, B 400 feet, and H 10 feet, we obtain $h = 3{\cdot}8$ feet.

The conditions under which this formula is applicable should, of course, be strictly stated, and they are as follows:—"When the piers are high enough to screen the inner area from the wind, when the depth is tolerably uniform, the width of entrance not very great in comparison with the section of the wave, and when the quay walls are vertical or nearly so, and the distance not less than 50 feet from the mouth of the harbour to the place of observation." [1]

The entrance width, however, is subject to other and further considerations. In tidal harbours there is the outrun of the ebb tide with the cumulative effect of the discharge of any upland waters, all tending to produce a rapid current in a narrow water way. And while the scour induced by this means is beneficial within certain limits in maintaining a deep channel, yet, carried to excess, it is likely to prove prejudicial to the stability of walls and piers by undermining their foundations, and, moreover, the rate of flow may be such as to interfere with and possibly prevent safe navigation. A velocity of from 3½ to 4 knots should be looked upon as the maximum current permissible.

Such, in brief compass, are some of the more important matters bearing on the general question of harbour design, from which it will be seen that there are many weighty considerations which contribute towards a determination of the proper form and arrangement of areas reserved for the reception of shipping. In the ensuing chapters, it will be our duty to investigate some of these features more closely and in greater detail.

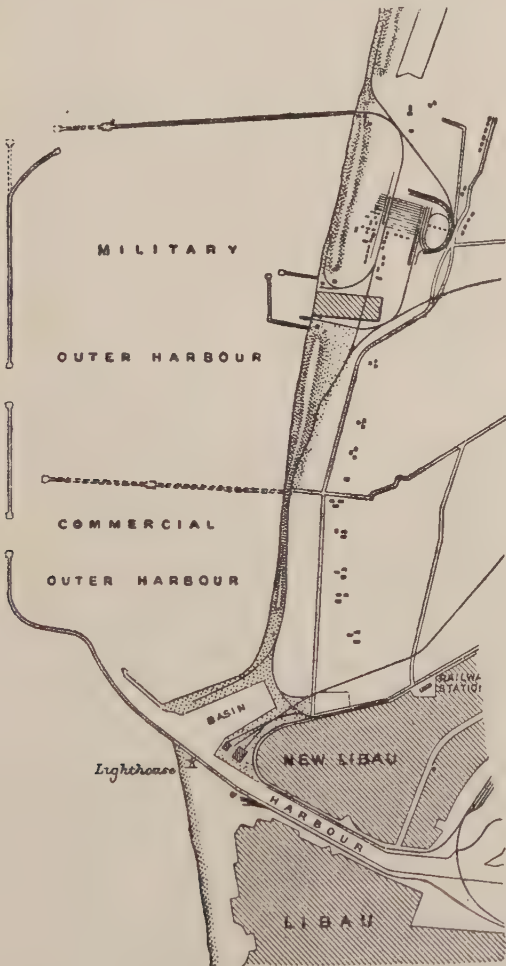
Meanwhile, we conclude the present section with a brief description of a few ports selected as furnishing fairly representative examples of the three principal types of harbours—viz., national harbours, commercial harbours, and fishery harbours, and also of a trio of harbours remarkable more for their form than for their size, and possessing interest out of proportion to their commercial importance.

The Military Outport at Libau.[2]—Libau, as a commercial harbour, dates from the thirteenth century, and various extensions have been made in its

[1] Stevenson on *Design and Construction of Harbours*, Third Edition, p. 165. *Cf.* also Gaillard on *Wave Action in Relation to Engineering Structures*, p. 89, for instances of reduction of waves in Duluth Harbour, U.S.A.

[2] Jarintzoff on "The Military Outport of Libau," *Min. Proc. Inst. C.E.*, vol. cxxvi.

accommodation from time to time. In 1870, when the Libau-Romen railway was constructed, the port came into considerable prominence, and in 1887 the Russian Government determined to make also a military harbour, the site of which was to be immediately to the north of, and connected with, the commercial harbour.



Scale, 1 inch = 5,000 feet.

Fig. 33.—The Port of Libau.

"In designing the general arrangement of the sheltering constructions of the outport, two questions had to be taken into consideration: (1) to lessen as much as possible the risk of the entrances and of the interior of the port being silted up by the coast drifts; (2) to prevent floating ice from accumulating in front of the walls and to assist the escape of the ice formed within the basin. For both purposes the design adopted may be considered as the

most suitable. The movement of the sand along the coast is of a two-fold character. In shallow water the sand is carried by the waves along the shore and accumulates at each exposed point, which tends to prevent its further movement. For that reason the more the southern mole of the commercial port was extended into the sea, the more rapid was the growth of the coast in the angle between the mole and the shore ; but, in the future, this growth will be slower, first, because the depth of the sea increases further from the shore, and secondly, because the mole was built out at once to a considerable distance and to a great depth, which obliged the waves from the west and south-west to glide along the mole and dash against the coast, thus scattering the sand collected. Certainly this does not prevent the harbour from silting up, but the sand is carried for a long distance along the coast, and, therefore, the danger of accumulations at the entrance of the harbour is considerably diminished. Beyond the breakwater the movement of the sand is produced by the coast current, in which the particles of sand are suspended. If the currents do not meet with any obstacles, the greater part of the sand is carried along the coast and is left in sheltered places, and this action is favoured by the circumstance that the breakwater and the point of the southern mole form a straight line. As regards the ice, which generally moves backwards and forwards from north to south, the arrangement of the walls in one line is very convenient. There is nothing to stop the ice and give isolated masses time to freeze together under the influence of the cold coast winds. Consequently there can be no accumulation of large ice masses, and a strong ice-breaker can at all times easily make a way out of the port into the open sea. The ice in the harbour, broken up by the ice-breaker, passes without difficulty through the three outlets ; but this ice, owing to the mildness of the climate, is never so thick as to be a serious obstacle to the movement of the ships."

The military port, as formed, is 7,700 feet long, 7,000 feet wide, and occupies about 1,200 acres. Its natural depth is 14 feet at a distance of 1,400 feet from the coast, 22 feet at a distance of 3,500 feet, and it gradually increases to 29 and 30 feet as it nears the breakwater. The width of each of the three entrances is 700 feet, and the general depth seaward is 30 feet, though diminished in places to as little as 24 feet.

Madras Harbour.—The case of Madras Harbour is interesting as an example of development by process of trial and error. The earlier dispositions, although founded on the advice of competent authorities and on principles sound in themselves, failed, by reason of the inadequate data at disposal, to realise the expectations of their promoters. It has only been through the experience gained from the initial results and through the modifications introduced in consequence, that a difficult problem has at length been satisfactorily solved.

The commercial ports of India are not numerous, in spite of the enormous extent of its seaboard. They can almost be counted upon the fingers of one

To face p. 42.]

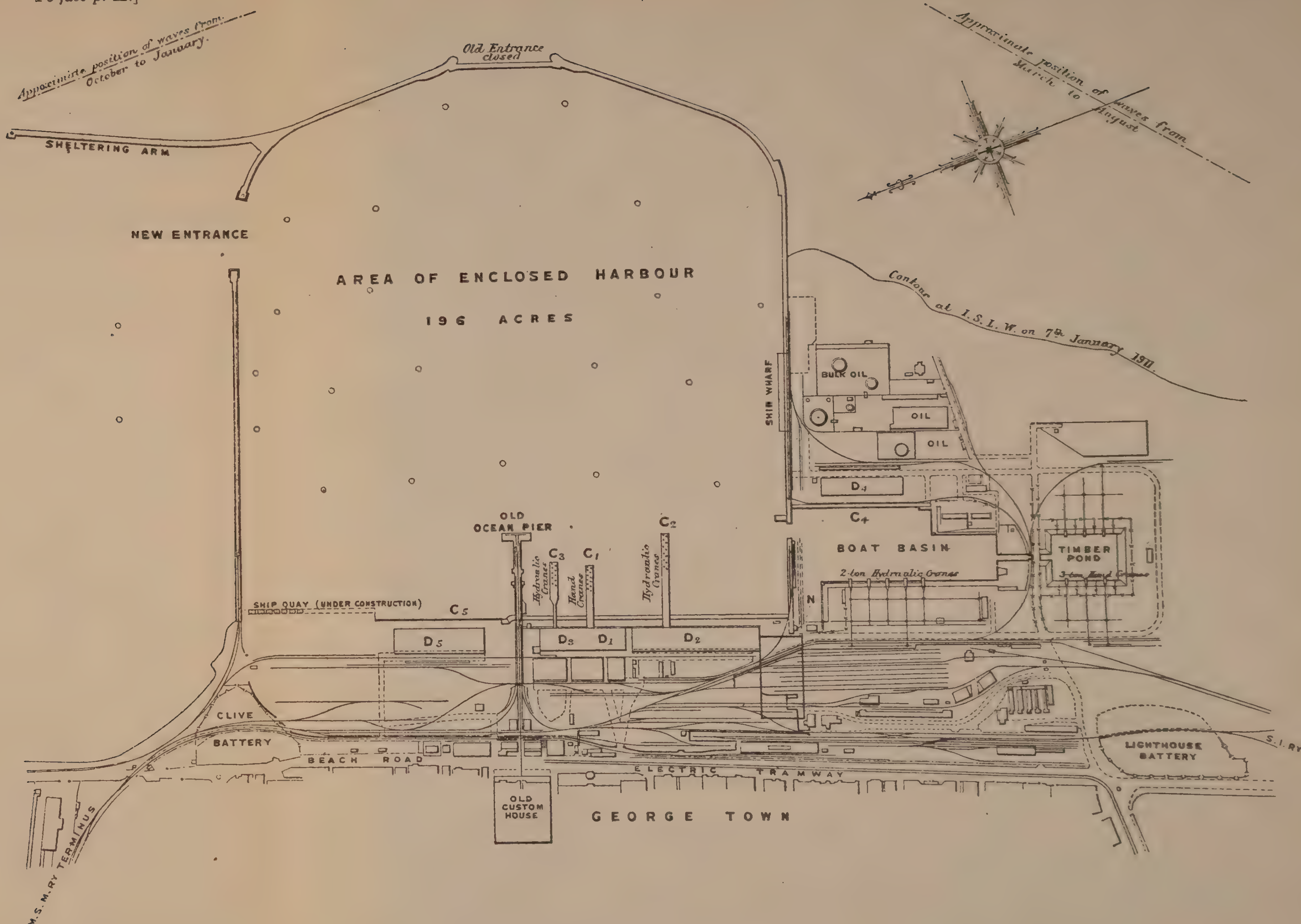


Fig. 34.—Madras Harbour, as remodelled. in 1911

hand, and most of them are of comparatively recent development. Up to the year 1875, there was nothing at Madras of the nature of a harbour, either natural or artificial; there was simply an open roadstead on an exposed, sandy coast, swept by storms and occasional monsoons of extreme violence, during which vessels were far safer out at sea. In fact, even at the present time, throughout the entire distance of 2,300 miles extending round the peninsula from Calcutta to Bombay, there exists no mainland port capable of affording adequate shelter to shipping in times of cyclonic disturbance; still less of accommodating it, uninterruptedly, at the quayside.

At Madras, the unloading of vessels was, until recently, entirely carried on by lighters, or barges, of about 2 to 10 tons capacity, which traversed

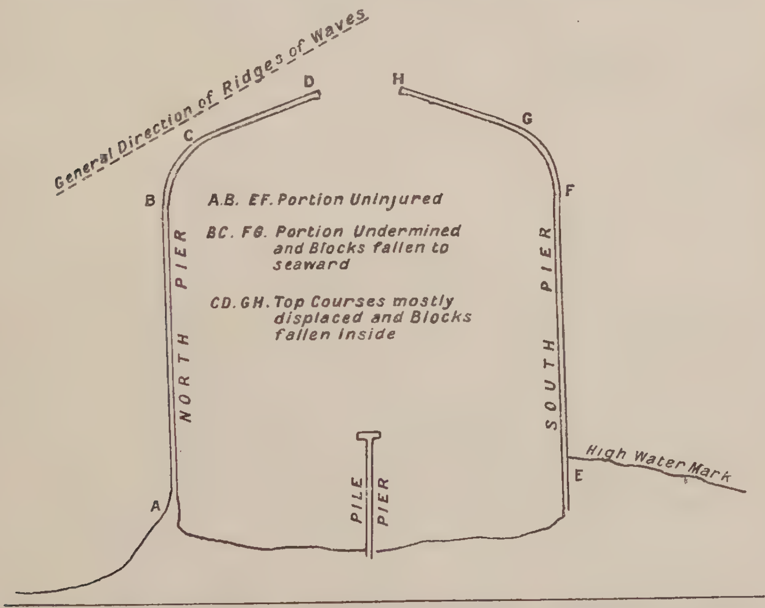


Fig. 35.—Madras Harbour. Sketch showing extent of damage done by storm of 12th November, 1881.

the distance between ship and shore. The transfer of cargo in the open was and could only be effected at considerable risk, and the first step to be undertaken in any attempt at an amelioration of the conditions was manifestly the creation of a sheltered area, within which barges could move freely and without danger.

There were no natural features to lend assistance to such an undertaking, and purely artificial dispositions had to be made. The initial design was prepared by the late Mr. Wm. Parkes, and consisted of two breakwaters projecting perpendiculary to the coast-line, with rectangular returns terminating centrally in pier heads, 450 feet apart. Subsequently, the plan was modified by the Marine Department, so that the return angles became

obtuse and the entrance width was increased to 550 feet. The arrangement as adopted is shown in fig. 35.

The north pier was commenced in 1875, and the south pier two years later. Unfortunately, as the work was carried out from the coast-line, the sand accumulated about it to the southward so rapidly as to cause the line of foreshore to keep pace almost with the work. It was only by pushing forward with despatch in the intervals between the monsoons that the walls were eventually got ahead of the sand drift. They reached their respective pierheads by the year 1881.

At this epoch a disastrous cyclone occurred. On 12th November, 1881, the sea swept over the breakwaters from both sides of the harbour, damaging the work to an enormous extent. Blocks of 27 tons apiece were dismantled and flung into the inclosure, the walls were undermined by the scour (in places to the unprecedented depth of 22 feet below water) and some of the work even fell outwards under the pressure of the water pent up within the harbour.

In consequence of this disaster, the walls, after repair, were raised an additional 9 or 10 feet in height, and this has so far proved satisfactory, for no case of overflooding has since occurred.

The incident afforded an opportunity for a review of the design with the object of ascertaining whether any modifications were desirable. In 1883, a local committee convened at Madras to consider the official report of Sir John Hawkshaw, Sir John Coode, and Professor Stokes, recommending a basis of reconstruction, advocated the adoption of an improvement scheme of their own, with a new entrance facing north-east. In assigning their reasons for wishing to abandon the original entrance, the members stated that:

"No matter what the direction of the wind, the unceasing swell on this portion of the coast rolls in with the crests of the waves parallel, or very nearly so, to the coast-line. In no case is it believed that the angle exceeds 30° to the general line of the coast. The result is that seas enter the present mouth freely, and, owing to the small length of the harbour, are not dispersed before reaching the shore at its base. The action is, of course, greatly intensified during storms, and particularly with the wind from the east. At such times, the sea inside the harbour, though not so high as outside, is certainly of a dangerous character, being exceedingly broken. Taking these and other facts into consideration, the committee have to record their opinion that unless means be found for closing entirely the present entrance, no radical cure will have been applied to the chief defect of the work as at present designed." [1]

In 1887 they issued a further statement.

"It is agreed on all hands that, owing to the frequently disturbed state of the water, the facilities for landing and embarking passengers, cargo, etc., offered by the harbour, are very much restricted, nor would it be feasible, for

[1] *Official Papers, Madras Harbour*, 1902, p. 39.

the same reason, to use, without serious interruption, wharves or jetties along the shore-line, or to keep in safety within it such improved lighters, tugs, and other harbour craft as would greatly increase its value as a trading port. Much cargo is said to be lost overboard in the process of transhipment, and, for want of tugs, no sailing vessels use the harbour at all. [1]

"The present or east entrance we believe to be the easiest and safest for ingress or egress, but not only does it admit the sea in the manner described, but we are of opinion that the time is not very far distant when the depth at the entrance will be so far reduced as to become too shallow for the larger class of vessels frequenting the port.

"The alternative is an opening in the north-east corner with a covering arm. This is the plan favoured by the Madras Board, and to this we have given our most careful consideration.

"The opinions of the captains of steamers frequenting the port differ materially. Some see considerable difficulty and danger in taking an entrance so placed; others see none. We give it as our opinion that, although it may not be so easy of ingress, and ships may be detained outside more frequently than at present, the increased difficulty is not sufficient to condemn it." [2]

The recommendation was, however, set aside by the Secretary of State, and the work proceeded in accordance with the original design.

The allusion to the silting of the eastern entrance indicates another difficulty of the situation. Both prior to and since the completion of the work, the entrance had shown manifest indications of shoaling, at the rate of about 1 foot per annum. Although disquieting, this was not a cause of immediate anxiety, in that there was still, as late as 1907-8, something like 34 feet depth of water to meet the requirements of vessels which did not reach that draught by 10 feet or more. Still, in view of future increments, the matter called for attention. Dredging operations were put in hand and the evil was temporarily checked, one month's work sufficing to remove a year's deposit.

Although shelved for the time, the project of a north-east entrance revived, and in 1902 practically the same recommendation as that put forward in 1883 was endorsed by an advisory committee, consisting of Sir George Nares, Sir Charles Hartley, and Sir William Matthews. Fresh proposals, including a covering arm, 1,200 feet long, were submitted, together with estimates based on the advice of Sir Alexander Rendel, obtained during a visit to Madras. After some discussion these proposals were at length officially approved. The work was duly put in hand and completed by the year 1911. The plan of the remodelled harbour is shown in fig 34. [3]

[1] *Official Papers, Madras Harbour*, 1902, p. 53. [2] *Ibid.*, p. 54.

[3] Unfortunately, since the above was written, it has to be recorded that during a cyclone of exceptional severity on the night of 22nd-23rd November, 1916, the monolith pierhead, 42 feet square and 50 feet in height, weighing complete with its lighthouse

The execution of the work brought about none of the evils gloomily prognosticated by its opponents. The actual result of the operation is that the harbour is now smooth enough for working cargo into and out of lighters, alongside the ships and piers, in practically all weathers.[1] Furthermore, the steamers themselves are enabled to lie alongside the wharves with scarcely any interruption from meteorological causes. There has also come into existence a fleet of 40- to 60-ton lighters, largely superseding the old 2- to 10-ton lighters, and these enable loading and discharging operations to be carried on much more conveniently and expeditiously than formerly.

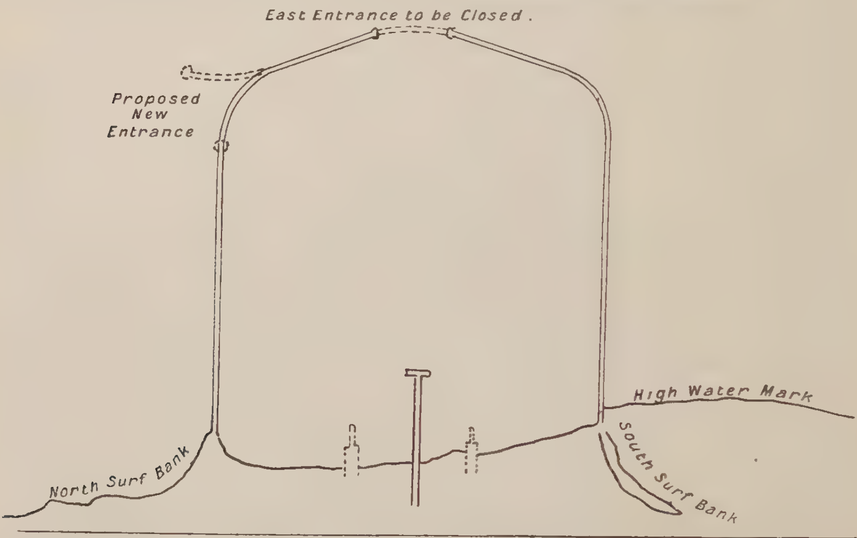


Fig. 36.—Madras Harbour. Proposals by Madras Special Committee, 1887.

For much of his information respecting the Port and its history, the author is indebted to the courtesy of Sir Francis Spring, K.C.I.E., the chairman and engineer of the Harbour Commissioners. The following interesting observations are direct from the pen of that gentleman :—

"At the present date,[2] 43 years after the commencement of the discussion, it is interesting to note how certain matters that came under notice have changed in relative importance in the estimation of those interested. Thus,

about 5,000 tons, and founded on a rubble bed on the sea bottom, which, at this part lies at a depth of 42 feet below low water, was tilted over into a hole about 10 feet deep caused by the scouring action of the storm waves on the northern and western sides of the base. Deprived of the protection afforded by the pierhead, a contiguous length of 130 feet of the sheltering arm was destroyed piecemeal. The cost of making good the damage will, it is feared, be considerable, but happily for present needs, the accident has not appreciably affected the character of the harbour for smoothness of water.

[1] That is, during such conditions as admit of vessels lying in the harbour, which, of course, is still as unsheltered as ever from wind pressure.

[2] 1911.

in the early days, the question of the value of the harbour as a shelter for shipping during storms, bulked largely with the protagonists. Lists were given of all the wrecks and losses of men and goods since 1746, and it was stated that in the 26 years ending in 1868, the vessels wrecked numbered 42, of an average tonnage of 480, involving the loss of 196 lives and of property worth £380,000. Again, in 1875 the Secretary of State wrote : ' Many human lives are sacrificed in the tempests which annually rage on the Coromandel coast, and it is probable that a large proportion of these might be saved if the vessels which are now surprised in the roadstead could seek the shelter of a harbour.' The fact really was that the port was then used by sailing ships of small tonnage for the most part, and it has to be recognised that conditions change greatly when 4,000 to 10,000 ton vessels, in smaller numbers, replace the swarms of little sailing ships of the old days. The problem of the present day is not how to afford shelter to vessels in stormy weather, but how to calm the water in which they are moored so as to enable them to work their cargo easily overside from or into lighters or at quays. Even as late as 1873, Mr. Parkes did not seem to recognise that a wave of 2 feet 7 inches, the height to which he proposed, by means of the harbour, to reduce the 15-foot wave, was a very formidable thing indeed from the landing and shipping point of view, especially with small lighters having a capacity between 1 ton and 10 tons. Now that the harbour has been remodelled so that heavy seas are no longer found inside it, the problem of the safety of the vessels resolves itself into a question of the pressure of the wind on their sides and the adequacy of their fastenings to withstand such pressure.

" Ordinarily, all vessels in the harbour are moored head and stern, and this was found to be an important factor in the problem to be dealt with by those who had to decide whether the cost of remodelling was necessary or not. For no matter what may be the direction of the wind at Madras, the lines of waves approaching the shore have a general tendency to parallelism with the shore. Much of the delay in arriving at the decision to alter the entrance of Madras harbour is now seen clearly to be ascribable to the fact that some of those consulted, and whose opinion carried weight with the authorities, failed to realise that, close to the shore on the Coromandel coast, the trend of the waves does not conform to the direction of the wind, in the same way as it would in the open sea. A ruler laid on the map along the lines of the waves may sometimes be about 30° out of parallelism with the general lie of the shore in one direction, and sometimes about as far in the other direction. Both of these sorts of waves, and, of course, also all waves having a direction intermediate between them, used to roll into the harbour impartially through the old entrance. No waves ever run from the north to the south, or from the south to the north, or anywhere near these directions, even if a strong north or south wind should happen to be blowing. Therefore, so long as the old entrance was open, a vessel moored head and stern

in the harbour, if lying north and south, rolled whenever there was any sea to speak of, and found it dangerous and difficult to handle cargo overside. If she lay east and west, she pitched instead of rolling, and had perhaps less difficulty with her cargo. When an east or west wind happens to be blowing, a vessel moored north and south may have, say, a 200-ton strain on her bow and stern lines, and the result in a strong wind generally is that her stern lines break and she swings ; but if moored east and west, she will be all right so far as her fastenings are concerned. The reverse takes place when a north or a south wind is blowing. In this case it is the vessel moored head to east that will break her stern lines and swing. Neither kind of wind, north-south or east-west, will affect cargo arrangements appreciably, these being affected by the swell alone, as explained.

"The change of entrance in no way improves this matter of swinging due to breakage of stern lines. All it has done is to put an end effectually to the swell that used to interfere badly with the working of cargo overside when a vessel was either moored north and south, or having been moored east and west, broke her stern lines in a north or a south wind and swung north and south."[1]

Respecting the shoaling difficulty, Sir Francis Spring adds :—

"The sandy shore of the thousand or so miles of the eastern coast of India is continually being acted upon by surface waves dashing upon it, for part of the year with a north-western set, and for part of the year with a south-western set. The resultant of these two sets has a north-westerly trend, and the effect is that wherever an obstruction juts out from the shore —*e.g.*, such an obstruction as is offered by the harbour arm,—there is necessarily an accretion of sand to the south of it, and the contrary, in the form of erosion, to the north of it. The accretion is greatly assisted in its formation by the effect of the wind upon the sand thrown up on the shore by the surface waves ; for it will be understood that in a tropical climate the sand is very quickly dried by the intense solar heat. The prevailing winds, blowing for months together in one direction, bear the dried sand landwards, and pile it up until its level is from 3 to 5 feet above the level of high water. Such an accretion, to the extent of some hundreds of acres, has formed itself on the south side of Madras harbour, affording valuable land for various port purposes.

"An interesting point presents itself in the consideration of the shore-line of the accretion. The theory has been put forward very tentatively that, when a line of coast is so orientated that the seasonal waves of greatest force and duration roll in on it at right angles, there will be less tendency for the sand to be driven sideways than when such waves habitually roll in obliquely. A glance at fig. 21 will show that close to the harbour, say within ½ mile of it, the accretion has nowadays so formed itself that its seaward edge is more or less parallel to the trend of the south-west monsoon waves.

[1] Spring on the Remodelling of Madras Harbour, *Min. Proc. Inst. C.E.*, vol. cxc.

It seems to be arguable that until the bay, farther south of this part, shall have filled so far that the parallelism in question is no longer to be observed, the tendency will be for the sand-creep eastward along the south harbour arm to proceed comparatively slowly. Indeed this theory conforms with the fact that slowness of forward creep has been evident for some years past. The total creep in 34 years has been 2,100 feet, or at the average rate of 70 feet per annum, whereas in the past 11 years the creep has been at the rate of only about 30 feet per annum. However, this argument must not be pushed too far, for part of the alteration of rate is doubtless ascribable to the deepening water met by the accretion as it works farther outwards." [1]

Fishery Harbour at Aberdeen.—Aberdeen is one of the largest fishing ports in the United Kingdom, being exceeded only by Grimsby in the quantity of fish annually landed upon its quays, and it affords a typical example of a harbour with special facilities for the fishing trade. The 29th Annual Report (1910) of the Fishery Board for Scotland remarks :—" Aberdeen has ever been prominent in the introduction of improved methods of fishing, of improved construction and equipment of fishing boats, and of improved facilities for the discharge, sale, and distribution of the enormous and ever-increasing quantities of fish landed." Fig. 36*a* is a plan of the harbour.

Prior to 1885 the chief varieties of fish brought to market at Aberdeen were herring, haddock, cod, whiting, flounder, and skate, and these were landed entirely from sailing boats, fishing mostly inshore. At the present time there are 64 varieties of fish, including haddocks, cod, herrings, ling, saithe, whiting, halibut—the finest halibut in the country are landed at Aberdeen—soles, plaice, and skate, caught and landed at the port by a fleet of steam fishing vessels. In the early days of the industry, plentiful supplies were found in the North Sea at no great distance from Aberdeen, and small vessels of moderate power were quite suitable for the work, many old paddle tug-boats being pressed into the service. As time went on, however, new fishing grounds were opened up, and the supply of fish is now obtained from many parts of the North Sea and North Atlantic, as far as Faroe, St. Kilda, and Iceland. These great distances have demanded a larger and more powerful type of fishing vessel, and the largest trawler of to-day has about three times the tonnage and twice the speed of the boats first employed at the port, and they carry about ten times the quantity of fish.

The quantity of coal required to keep the Aberdeen fleet at work on the fishing ground is about 334,000 tons per annum, or about half the total quantity imported, and four large ice factories are kept running continuously night and day all the year round to supply the trawlers, the total quantity manufactured in one year being about 120,000 tons.

In addition to the traffic in wet fish for immediate despatch—which is about one-half the total catch—there are over a hundred fish-curing factories in the vicinity of the fish docks, which produce enormous quantities of

[1] *Min. Proc. Inst. C.E.*, vol. cxciv., p. 165.

"Aberdeen Finnan Haddies" and "Kippered Herrings" for the southern and foreign markets. A considerable foreign trade is also done in the curing of cod, ling, saithe, haddocks, and similar round fish, by drying them in the air, after being cleaned, split open, and salted.

During the summer months herring fishing is carried on at the port, as many as 400 fishing boats being employed in the season. Until recently these have been entirely propelled by sail, but now the application of steam and oil motors has revolutionised the industry. The port is now the headquarters of a fleet of "Herring Drifters." The herrings are cured in barrels, about 100,000 being shipped from the port in a season.

The regular fishing fleet consists of 217 steam trawlers, 53 steam line vessels, and 77 sailing boats, and these are manned by about 3,000 fishermen, while the persons connected with the industry ashore number about 10,000.

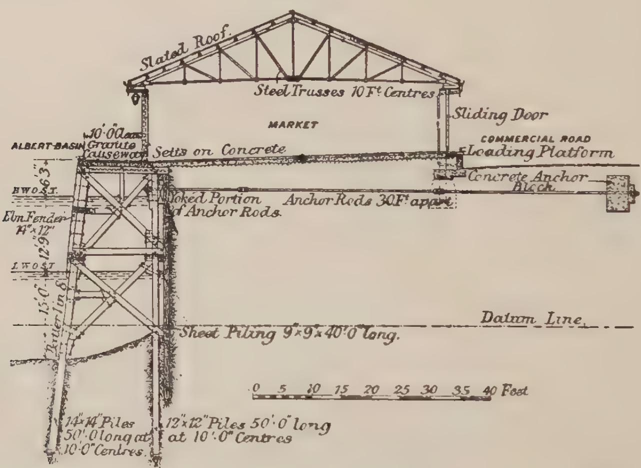


Fig. 36*b*.—Cross-section of Fish Wharf and Market at Aberdeen.

The value of the fishing vessels and gear belonging to the port is over a million sterling, and the capital invested in the industry ashore, including fish docks and fish market, curing establishments, ice factories, workshops, stores, etc., is estimated at about two millions more, making three millions in all.[1]

For the following notes on the special fishing facilities provided at the port, the author is indebted to the Harbour Engineer, Mr. R. Gordon Nicol :—

The white fishing industry is accommodated in the Albert Basin, more particularly in the western portion from about the east end of the Graving Dock. The area of the Basin is about 21½ acres, and the quayage about a mile. The market extends along the north, west, and a portion of the south sides of the Basin, and has a total frontage of 2,295 lineal feet, and an area of 2½ acres. The wharfs have been erected by and belong to the Harbour Commissioners, and the market belongs to the Town Council. Both have

[1] The figures given relate to the period just prior to the outbreak of the war.

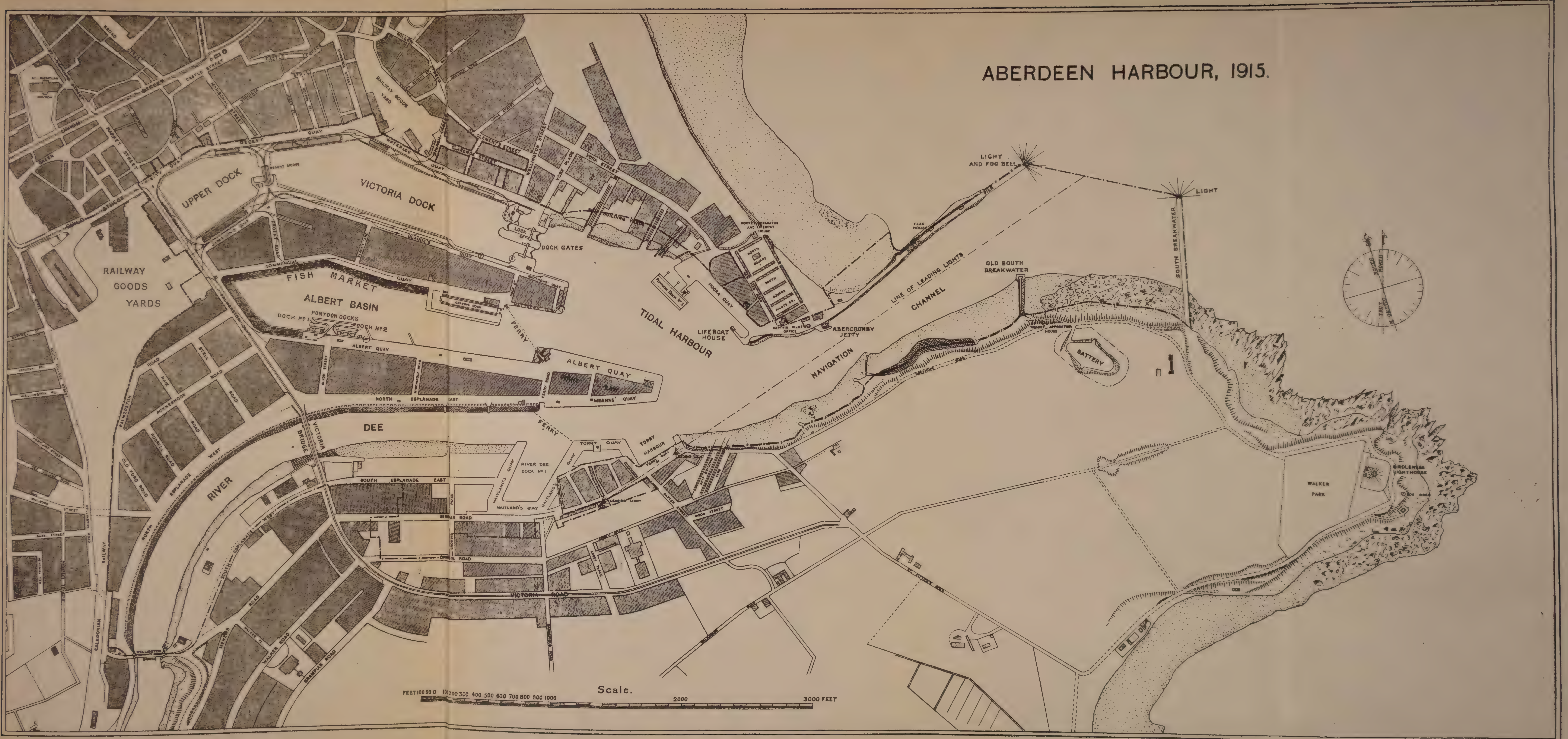


Fig. 36a.—Aberdeen Harbour.

been erected in parts at various times, and the cross-section shown in fig. 36*b* represents one of the most recent portions. The wharf is constructed of pitchpine with elm fenders on the face; the deck is of rolled steel joists and concrete, overlaid by a causeway. The floor of the market, generally, consists of concrete, and is sloped from the back to the front to allow of easy cleansing.

From 30 to 70 trawlers land their catches per day. The vessels arrive at the Fish Market, discharge and proceed to the south side of the Basin for supplies of coal, ice, and stores, ready for the next voyage. On account of the increased pressure on the available space, the more recently erected portions of the market have been provided with ramps to the street outside, so that carts laden with stores of coal and ice may enter the market and load the trawlers while lying alongside the market. There is no mechanical plant for loading coal or ice.

The Albert Basin is accessible from the sea at all stages of the tide, and there is sufficient depth of water in the entrance channel to allow trawlers to enter except at low water of spring tides. A large quantity of rock in the navigation channel is in course of removal in order to give increased depth of water.

The herring fishing at Aberdeen was formerly accommodated at the whole of the south side of the Albert Basin, but the increase in the white fishing has led to the former being limited to the eastern portion of the south side. Additional accommodation has also been provided on the north side of the River Dee, and Mearns Quay, of which an extension is in progress, was constructed for this purpose. On the south side of the river a new dock, called the River Dee Dock, No. 1, has lately been constructed for the landing of herrings, and also for the wintering of herring drifters.

Three pontoon docks are available in the harbour: No. 1 of 425 tons deadweight lifting capacity, No. 2 of 600 tons, and No. 3 of 5,350 tons. The two smaller are kept constantly employed in dealing with fishing vessels and the smaller class of merchant vessels. The largest dock is arranged to lift six drifters, or four trawlers, or one large vessel.

Whitby Harbour.—The town of Whitby, lying at the mouth of the River Esk, affords an instance of a fishing harbour maintained chiefly by tidal scour. There are two piers at the entrance, the west pier originally projecting considerably further seaward than the east pier. The meeting of the flood-tide, however, with the river current, produced an eddy just within the west pierhead, leading to slack water and shoaling.

Thus, a vessel desirous of entering the harbour had to go nearer the east pier in order to avoid the bar; in so doing it was in danger of losing steerage-way, owing to the strength of the flood-tide (which flowed eastward at a considerable rate), and tended to drift beyond the east pierhead before making the entrance. In order to minimise the trouble of the bar, some large stones were placed N.N.W. of the west pierhead below the water level, at which

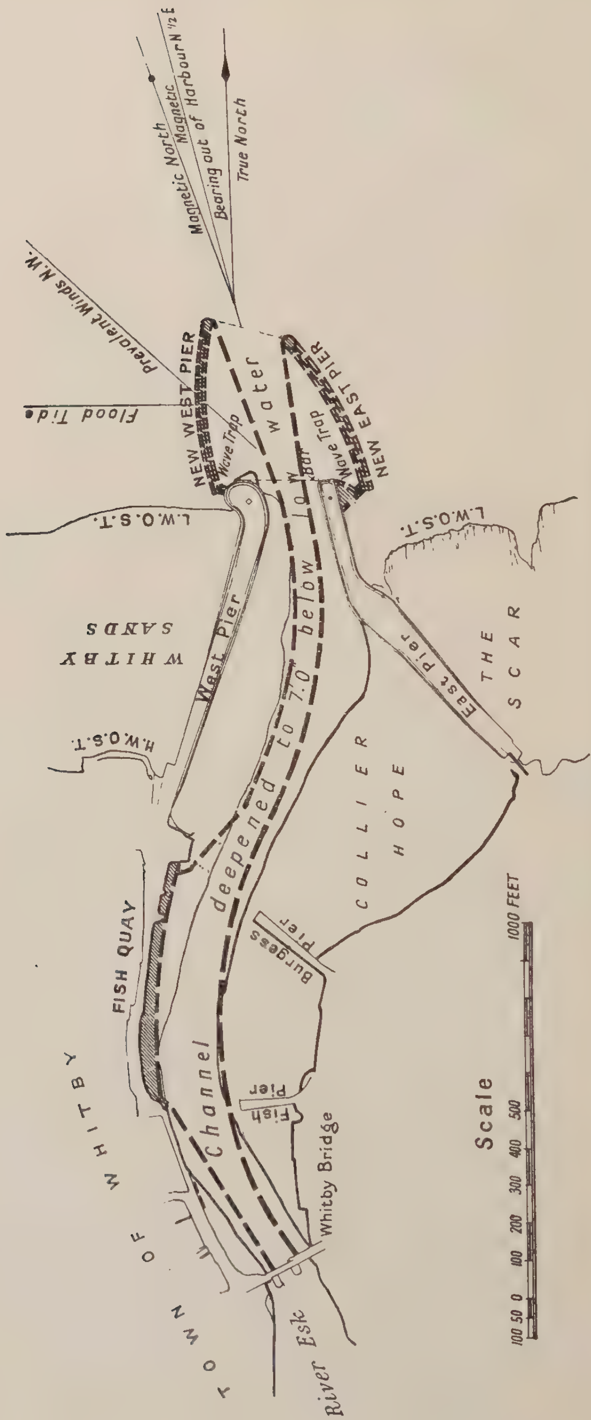


Fig. 37.—Whitby Harbour. Plan showing improvements.

ships might safely enter the harbour. These stones acted as a groyne, tending to prevent the formation of the bar by arresting the eastward progress of the sand deposit.[1]

The remedy subsequently adopted consisted in prolonging the east pier until both pierheads were in a line parallel to the set of the tide. By this means the width between the pierheads was diminished by nearly one-half, the bar disappeared for a time, and heavy seas which formerly entered the harbour were perceptibly reduced. The projection of the east pier, however, caught the waves from the north-west, instead of allowing them to pass the entrance as before. These waves, passing into shallow water, stirred up the sandy bottom, becoming heavily charged with material. They swept along until they struck the inner face of the east pier extension, whence they rebounded within the harbour, and, reaching slack water, the sand which they carried was deposited.

Besides silting up the harbour, the decreased width of the mouth made the entrance exceedingly dangerous, as, owing to the rapid cross-flow of the tide, a vessel had great difficulty in shooting in between the pierheads when running before a north-west wind. She might strike the east pier-end or drift on to the rocks beyond; or, if she effected an entrance, she might collide with the inner face of the extension.

The state of affairs, therefore, was far from satisfactory, and a further scheme of improvement was undertaken. Prolongations of the piers were made into the sea to a permanent depth of 7 feet at low water, and these were so designed as to form wave traps on each side, reducing the admission of waves and consequently the range of sea within the harbour. Between the piers and up the River Esk as far as Whitby Bridge, which lies beyond the new Fish Quay, a channel has been dredged to give a depth of 7 feet at low water. In front of the Fish Quay, which is 700 feet long, this channel is widened into an embayment which extends right in to the face line of the quay.

These works have been carried out as shown in fig. 37.

The prominent position of Whitby gives it a great advantage for sailing boats over embayed harbours, such as Scarborough and Hartlepool, where boats often lose much time by being becalmed; while at Whitby, when there is any wind at all, boats get it immediately outside of the pierheads.

The position of the railway at Whitby, alongside of the harbour and at the level of the quays, is another great advantage as compared with North Shields, Scarborough, and other harbours, where fish have to be carted uphill at great expense, to the railway.

Danish Island Harbours.[2]—The Danish Isles and the Peninsula of Jutland have an area of only 14,850 square miles and a shore-line of about

[1] *Vide* Austen on Whitby Harbour, *Min. Proc. Inst. C.E.*, vol. clvi., p. 264.

[2] P. Vedel on "Island Harbours," *Trans. Am. Soc. C.E.*, vol. liv., Part A., Proc. Int. Eng. Conf., 1904.

3,274 miles. On a part, perhaps about one-seventh, of this length, particularly on the south coast of the Island of Bornholm in the Baltic, on the north

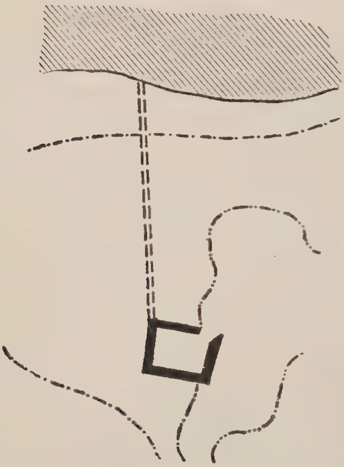


Fig. 38.—Plan of Arnager Island Harbour.

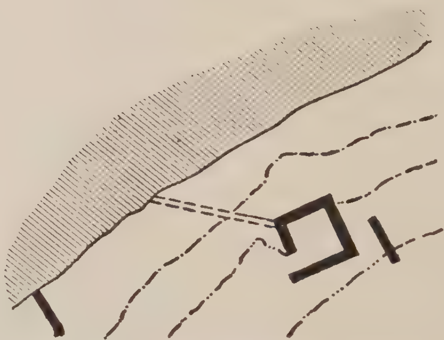


Fig. 39.—Plan of Snogebœk Island Harbour.

Fig. 40.—Plan of Hundested Island Harbour.

coast of Zealand facing the Cattegat, and on the north and west coasts of Jutland, facing the Skagerrack and the North Sea, construction of harbours is rendered difficult by the littoral drift. Here the island harbours are

situated, to which allusion has already been made. Two of them—viz., those at Arnager and Snogebœk on Bornholm were built in a tentative way in 1883 and 1888, whereas the third, at Hundested on Zealand, was formed in 1893 by the transformation of an originally land-connected harbour. All three, shown in figs. 38-40, were built by Mr. H. Zahrtmann. The basins, inclosed by riprap moles, are from 4 feet 6 inches to 8 feet deep, and cover areas of ·27, ·2, and 1·62 acres respectively, or, including the outer basin of the latter, 2·16 acres. From the moles, open viaducts, wooden or composite, from 330 feet to 660 feet long, lead to the shore, and are divided into from thirteen to twenty bays, which span openings from 20 feet to 30 feet wide.

On the coast of Denmark, tides are insignificant, hardly perceptible in the Baltic, not exceeding 1 foot in the Cattegat, and rising only to 4 feet 6 inches in the North Sea near the Southern Boundary. Hence, the movement of material which takes place cannot be due to tidal action, but to the action of the waves, combined with that of local currents, and attributable, therefore, to the effect of the wind.

The object aimed at by the three fishing ports seems to have been accomplished fairly well. At neither Arnager nor Snogebœk has material accumulated to an alarming degree; it is pure quartz sand, the size of the grains being ·45 and ·25 mm. respectively. At Hundested the drifting material is more heterogeneous, consisting of a mixture of quartz sand with grains of ·25 mm. in diameter, gravel, shingle, larger pebbles, and good-sized boulders. Some accumulation has taken place inside the 5 m. contour, and banks have formed at the south-east mole and at the shore south-east of the port; but a state of equilibrium seems to have been reached, in which these two banks play an important part.

CHAPTER III

THE TIDES.

Tidal Origins—Newton's Theory—Laplace's Theory—Tidal Nomenclature—Diurnal Inequality—Tidal Wave—Establishments of Ports—Tide Tables—Tide Prediction —Kelvin's Predicter—Tidal Ranges—Exceptional Tidal Phenomena.

In addition to perplexities arising out of the vagaries of coastal currents, the harbour engineer is confronted with a fresh series of problems due to a class of physical phenomena which are closely allied to currents and, indeed, give rise to certain of them. These are the tides.

Tidal Origins.—It is a matter of common observation on the sea coast, and particularly at straits and inlets, that the level of the sea undergoes a constant oscillation, rising and falling generally twice within the space of about 25 hours. Although, in early ages, the cause of this periodical rise and fall was to a large extent correctly surmised, it was only vaguely understood, and not until Newton established his laws of gravitation was a definite and convincing explanation forthcoming. It has now been clearly established that the phenomenon is due to the difference in the combined gravitational attractions of the sun and the moon upon various parts of the earth's surface.

The fact that high tides are invariably associated with times of new and full moon makes it evident that lunar influence is concerned in their production, and the inference is confirmed by the additional fact that times of high and low water occur about fifty minutes later every day, a period which harmonises with the average period taken by the moon to pass the same meridian on successive days.

The influence of the sun is not quite so apparent, but it is demonstrable from the same considerations as those which identify the phenomenon with the gravitational attraction of the moon. The moon's tidal influence is the greater, because the moon is nearer the earth. The sun's distance is nearly 390 times as great as the moon's distance, though, on the other hand, its mass, which is also a determining factor, is some 26 million times greater than that of the moon. The tidal force[1] varies directly as the mass and inversely as the cube of the distance. Thus we have, in round figures :

$$\frac{\text{Moon's influence}}{\text{Sun's influence}} = \frac{390^3}{26{,}000{,}000} = 2\tfrac{1}{4}, \text{ say.}$$

More correctly, the ratio is 2·34 to 1 or $\frac{7}{3}$.

[1]Tidal force must not be mistaken for gravitational force. The latter varies inversely as the *square* of the distance ; the former, which is the *difference* in the gravitational force exerted by the external body upon two points of the earth's surface, involves another power of the distance.

Mathematical presentations of the gravitational theory have taken several forms, of which the most notable are those of Newton and Laplace.

Newton's theory, known also as the **statical** or **equilibrium theory,** is based, as already stated, on the differential attraction exerted on a particle at the earth's surface lying on a joint axis of the moon and the earth, and one situated on an axis at right angles to this, or, what amounts to the same thing, at the earth's centre. The distance apart of the centres of the earth and the moon is 60 times the earth's radius. Taking a diameter of the earth which, extended, passes through the moon, the ratio of gravitational force at three points—one at each extremity of the diameter and one at the centre—is—

$$\frac{1}{59^2} : \frac{1}{60^2} : \frac{1}{61^2},$$

and the differences of these, which give the ratio of attraction at each extremity of the diameter, are

$$\frac{1}{59^2} - \frac{1}{60^2} = \frac{1}{3,481} - \frac{1}{3,600} = \frac{1}{105,307}$$

and $$\frac{1}{60^2} - \frac{1}{61^2} = \frac{1}{3,600} - \frac{1}{3,721} = \frac{1}{110,708}.$$

The mean is $\frac{1}{107,940} = \frac{2}{215,880}$, say $\frac{2}{216,000} = \frac{2}{60^3}$, which, although it represents a differential attraction too small to have any appreciable effect on the solid crust, suffices to disturb the more mobile particles of the sea.

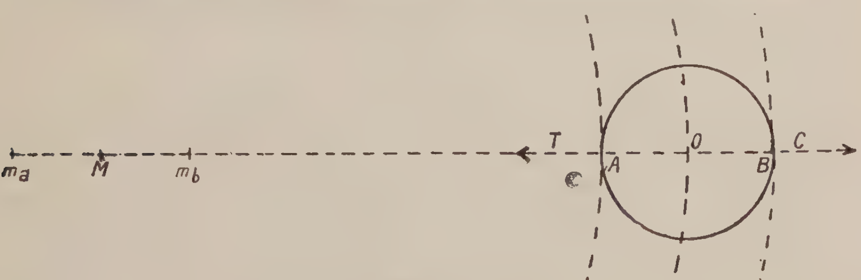


Fig. 41.

The following illustration will perhaps help to make the matter clearer. Let us take the case of a solid body A B, moving in a circular orbit about a centre M, the attraction of gravity T being just sufficient at each instant to neutralise the centrifugal force C. Now, assume the body to revolve round M without rotation, so that the point A describes a circle about m_a, and the point B, a circle about m_b. Then every particle of the body A B is moving, not concentrically, but with constant velocity, so that the action of centrifugal force is everywhere equal and parallel. But the gravitational pull varies inversely as the square of the distance, and is greater at A than

at B. Consequently, at A it exceeds the centrifugal force, while at B the centrifugal force is greater. In other words, there is at A a resultant force towards M, and at B a resultant force away from M, while at O, the mean position, the centrifugal and gravitational forces cancel one another.

In applying this illustration to the earth, revolving round the gravitational centre of itself and the moon,[1] let us recall the fact that the earth's rotation is performed once every 24 hours, and that at the end of each of these periods the earth is found in the same relative position to the moon as if it had not rotated at all. No difference, therefore, in the forces at work is brought about by the rotative action.

It must not be inferred that there is any actual vertical movement or lifting of the water due to the tide generating force, which, in itself, is insufficient to overcome gravity. The only direct effect is that of making the water appear slightly lighter, or heavier, as the case may be. The raising of the level of the water is due to the **horizontal** movement which sets in towards the points immediately beneath the moon at the extremities of the earth's diameter. In fig. 42 the arrows A A represent the direct generating pull along this diameter or axis, and B B the corresponding depression which is produced along the whole of the circumference of the circle passing through B B at right angles to the axis A A, and, therefore, perpendicular to the plane of the paper. Then between the point A and the great circle B B the tide generating force assumes a series of intermediate directions, and at certain points lying on a circle C C parallel to B B is tangential to the earth's surface—that is, it is entirely horizontal at these points. The components of the force in various positions, resolved parallel to the surface, cause a general movement of the water towards A A and away from B B, and this movement varies in intensity, being greatest at C and diminishing to zero at A and B.[2]

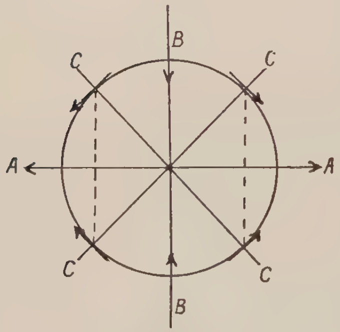


Fig. 42.

Newton's theory assumed the equilibrium, at each moment, of the particles under the forces acting upon them. This, of course, is incorrect, or, rather, it is an incomplete presentment of the facts. Under the equilibrium theory it would be a legitimate conclusion that high water should coincide with the passage of the moon across the meridian, but this is not the case[3]:

[1] This point is situated 3,000 miles from the earth's centre, and, therefore, 237,000 miles from that of the moon.

[2] For a more complete explanation of the tide-generating force, the reader is referred to Chapter V. in Sir George Darwin's work on *The Tides*, 3rd edition, 1911.

[3] Newton, as a matter of fact, deduced the exact contrary in a certain special case in which he based his investigation on the assumption of a narrow equatorial belt of water. His conclusion only holds good so long as the depth of the belt or canal does not exceed

it is some time later, generally several hours. The discrepancy, in fact, in some places in the Pacific Ocean (where the tidal wave has free scope) is such that it is often low water at the time of the moon's transit. To some extent, therefore, the explanation is evidently inadequate. Laplace, in his investigation, went a step further and took into account the disturbance due to the earth's rotation, and his theory is distinguished, accordingly, as the **Dynamical Theory.** In it, the movement of the water is calculated as the resultant of the diurnal revolution of the earth and the moon's attraction. Neither theory accounts for all the phenomena presented, and both involve assumptions of doubtful validity, but, in general, it may be said that they serve as a sufficiently satisfactory basis of approximate explanation for a problem which is too full of complexities to admit of complete solution. Other theories, on the same basis, but with additional modifications, have been put forward, but they are of interest merely from an academical point of view, and it is not proposed here to pursue the matter further. It is sufficient for our purpose to accept the phenomena as they exist, and to confine our studies to the conditions under which they occur.

Tidal Nomenclature.—Tides are distinguished as Springs and Neaps.

Spring tides are the highest tides of the month, and they occur when the moon is new or full, or nearly so ; in technical terms, when the moon is in conjunction with or in opposition to the sun, and the tides raised by the two bodies are exactly in unison, so that their joint effect is a maximum. At such times the moon is said to be in **syzygy,** and the periods when high tides prevail are sometimes referred to as the **syzygies.** Owing to retardation arising out of terrestrial conditions, the highest tide for each place is not coincident with conjunction and opposition, but occurs at some constant interval after new and full moon, which is known as the **age** of the tide.[1]

Equinoctial Spring Tides.—At or about the time of the equinoxes, the sun and the moon are actually or nearly vertically over the Equator. Their influences are then most closely coincident and direct, and equinoctial spring tides are, therefore, exceptionally high.

Neap Tides are the lowest tides of the month, and they occur about the times when the moon is in her quarters, or technically, in quadrature. The delay due to the age of the tide at springs is experienced equally in regard to neaps.

Shortly before and after new moon, the sun and the moon occupy positions such that their resultant attraction on the surface of the earth is directed

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12 miles. As an interesting point, it may be added that the variation in the moon's motion, the cause of which is analogous to that of the tides, causes her orbit to be nearer the earth at new and full moon and farther from it at first and last quarters. This corresponds to the conclusion in the "canal" theory of the tides.

[1] Thus at London Bridge the age of the tide is 2½ days. On the East Coast of England, North of Harwich, and on the East Coast of Scotland it is 2 days. In the English Channel, round Ireland, and on the West Coasts of England and Scotland it is 1½ days.

towards a point somewhere between them. This causes high water to occur a little before and a little after its usual time,[1] and the tide is said to *prime* and to *lag*. Similar action takes place just before and after full moon. The average interval between corresponding tides on successive days is 24 hours 51 minutes, but the priming and lagging each make differences of from about 20 to 40 minutes in the tidal interval, so that it ranges between 24 hours 32 minutes and 25 hours 32 minutes.

Diurnal Inequality.—In some parts of the globe the difference in the height of the tides on the same day is very noticeable. This diurnal inequality, as it is termed, is due to the fact that the moon moves above and below the Equator, and at the beginning and end of a period of 12 hours (except when the moon is exactly over the Equator) a point on the earth's surface occupies different positions in regard to the plane of the moon's orbit and so comes under lunar influence in varying degree. The diurnal inequality is very marked in the Indian and Pacific Oceans, but in the North Atlantic it is only slightly evident. European tides, in fact, are much less complicated than those in other quarters of the globe.

Tidal Wave.—The rise in the sea level, which produces high water along the coasts of Western Europe, originates in the Atlantic, and travels at the rate of about 800 miles per hour. Though following, generally, the moon's transit over the meridian at an approximately definite interval, its course is at first easterly (*i.e.*, opposite to the moon's direction) and then northerly. It reaches the western coast of Spain and Portugal in about 2 hours after the moon's transit, the western coast of France in about 3 hours, Land's End and the western coast of Ireland in about 4 hours, the Hebrides in about 6 hours, and the Orkneys and Shetlands in about 8 hours. Then pivotting round the latter, it travels southward, down the North Sea, reaching midway about 12 hours, and entering the Straits of Dover nearly 24 hours after its inception. Here it encounters a branch of the wave, which, travelling up the English Channel, has passed through the Straits within 12 hours of the moon's transit.

Establishment.—The approximate constancy of the intervals between the times of arrival of the tidal wave at successive points in its course enables the time of high water at a port to be determined approximately by simple addition. The data required are the time of the moon's transit and the average interval elapsing between that and the time of high water. This interval is known as the establishment of the port, or more correctly the **mean establishment.** The establishment is strictly the clock time of high water, before noon on the day of full moon and after noon on the day of new moon. On Admiralty charts this time is designated by the letters: H.W., F. and C. (high water, full and change).

A table of mean establishments for the more important ports and harbours on the British coast and the North-Western ports of Europe is given below.

[1] The tide is late before new moon and early after it.

Establishments of Certain Ports.

	Time of High Water F. and C.			Time of High Water, F. and C.	
	Local.	Greenwich		Local.	Greenwich
	H. M.	H. M.		H. M.	H. M.
Aberdeen,	1 0	1 8	Havre,	9 9	9 9
Aberdovey,	7 51	8 7	Holyhead,	10 11	10 29
Aberystwith,	7 37	7 53	Hull,	6 29	6 30
Alderney,	6 46	6 55	Inverness,	0 5	0 22
Alloa,	3 18	3 33	Ipswich,	0 35	0 30
Antwerp,	3 45	3 27	Kinsale,	4 43	5 17
Appledore,	5 58	6 15	Kilrush,	4 42	5 20
Arbroath,	1 35	1 45	Kirkwall,	10 20	10 32
Ardrossan,	11 49	12 8	Lancaster,	11 16	11 27
Ayr,	11 50	12 9	Leith,	2 22	2 35
Banff,	0 28	0 38	Lerwick,	11 5	11 10
Bantry,	3 47	4 25	Limerick,	6 10	6 45
Belfast,	10 43	11 7	Liverpool,	11 23	11 35
Berwick,	2 18	2 26	Llanelly,	6 16	6 33
Blyth,	3 15	3 21	Londonderry,	8 1	8 30
Bordeaux,	6 50	6 52	London Bridge,	1 58	1 58
Boston (Linc.),	6 30	6 30	Lowestoft,	9 57	9 50
Boulogne,	11 22	11 16	Maryport,	11 26	11 40
Brest,	3 46	4 4	Middlesbrough,	3 47	3 52
Bridgewater,	8 0	8 12	Milford Haven,	6 6	6 27
Bristol,	7 18	7 28	Montrose,	2 17	2 27
Calais,	11 44	11 37	Newcastle,	3 26	3 32
Cardiff,	7 0	7 13	Newhaven,	11 14	11 14
Cardigan,	7 1	7 20	Newport (Mon.),	7 10	7 22
Carnarvon,	9 30	9 47	Ostend,	0 25	0 13
Cherbourg,	7 56	8 2	Padstow,	5 13	5 33
Coleraine,	6 24	6 51	Penzance,	4 30	4 52
Cork,	4 53	5 27	Perth,	3 35	3 49
Cowes,	10 15	10 20	Peterhead,	0 34	0 41
	11 15	11 20	Plymouth,	5 37	5 54
Cromer,	7 0	6 55	Portland,	7 1	7 11
Cuxhaven,	0 49	0 14	Portsmouth,	11 41	11 45
Dartmouth,	6 16	6 30	Preston,	11 20	11 31
Deal,	11 15	11 9	Queenstown,	4 58	5 31
Dieppe,	11 3	10 59	Rotterdam,	4 35	4 17
Douglas (I.O.M.),	11 12	11 30	Rouen,	2 28	2 24
Dover,	11 12	11 7	Sheerness,	0 37	0 34
Drogheda,	11 0	11 25	Sligo,	5 23	5 57
Dublin,	11 32	11 57	Southampton,	0 33	0 39
Dundee,	2 32	2 44		10 56	11 2
Dunkirk,	12 24	12 15	Stockton,	3 40	3 45
Falmouth,	4 57	5 17	Stornoway,	6 46	7 12
Fleetwood,	11 12	11 24	St. Malo,	6 0	6 8
Flushing,	1 25	1 11	St. Nazaire,	3 47	3 56
Fowey,	5 14	5 33	St. Peter Port,	6 37	6 47
Fraserburgh,	0 40	0 48	Sunderland,	3 22	3 27
Galway,	4 35	5 11	Swansea,	6 0	6 16
Glasgow,	0 30	0 47	Waterford,	6 6	6 34
Goole,	7 26	7 29	Westport,	4 57	5 36
Gravesend,	1 5	1 3	Wexford,	7 21	7 47
Greenock,	12 5	12 24	Whitehaven,	11 14	11 28
Greenwich,	1 42	1 42	Wick,	11 22	11 34
Grimsby,	5 36	5 36	Wicklow,	10 29	10 53
Hamburg,	4 55	4 15	Workington,	11 20	11 34
Hartlepool,	3 28	3 33	Yarmouth,	9 29	9 22
Harwich,	11 56	11 51	Youghal,	5 4	5 35

Tide Tables.—While the establishment is a sufficient guide for approximate purposes, and serves the mariner as a rough indication in regard to the time of high water, it is often desirable to obtain more reliable forecasts both as to time and height. With the aid of certain data and by means of formulæ which have been devised and brought to a high degree of perfection, it is now possible to predict the times and heights of high water for considerable periods, and many ports actually publish their local tide tables for twelve months in advance. The Admiralty also issue tide tables for a number of places in this country and abroad.

Tide Prediction.—The sun, the moon, and the earth occupy almost identically the same relative positions in the heavens every 18 years and 11 days. Consequently tidal phenomena are reproduced under similar conditions at the end of successive periods of this duration, known to the ancient Chaldeans, and called by them a *Saros*. If, therefore, there be obtained the records of a series of tides during one complete Saros, they will serve to establish a suitable prediction for any subsequent Saros.[1] These records are, however, not always, or even commonly, available. The predictions have generally to be made from certain local observations over a short interval of time expanded by a system of harmonic analysis.

The observations should extend over at least a fortnight, so as to include a spring and a neap tide. If there is no automatic tide gauge recorder available for the purpose,[2] the tidal readings should be booked, generally at intervals of an hour, but every 5 or 10 minutes about the time of high and low water, so as to obtain a fairly accurate representation of a curve, which should be drawn through a series of points set out on squared paper. The times of high and low water have then to be referred to the time of the moon's transit, and any irregularities in the interval between transit and high water should be carefully noted. The average interval from the moon's transit to high water of spring tides gives the mean or corrected establishment, and the average interval from full and change of the moon to spring tides gives the age of the tide.

In order to arrive at a basis of prediction for future tides, the mathematician finds it convenient to dissect the tidal curve and resolve it into a series of independent waves so proportioned as to produce the actual result by cumulative effect. For this purpose, he assumes a moon and a sun to move strictly along the equatorial circle, so causing regular tides; to these he adds the sum or difference of tides produced by imaginary satellites of various masses moving in planes above and below the equator. The orbits and masses of these satellites are so adjusted, in conjunction with the postu-

[1] The eleven odd days in the Saros cause the conjunction point to move forward 11° along the ecliptic; this also produces a change in the declinations, which may amount to 4½°. Hence some modifications are required in the tides of the earlier Saros if exact prediction is desired.

[2] A description of a tide gauge recorder will be found on p. 84.

lated equatorial track of the hypothetical moon and sun, that the combined effect is that of the real moon and the real sun moving in their proper orbits.[1] For the complete determination of the tidal curve some 30 pairs of numbers corresponding to 30 imaginary satellites are requisite, and these may be divided into three groups.

The first and most important group comprises the semi-diurnal tides following one another at intervals of about 12 hours.

The second group are diurnal tides of rather less height than the first group, and occurring at intervals of about 24 hours.

The third group is least important, and consists of tides due to occasional interferences: they occur at intervals of about 3, 6, and 8 hours, a fortnight, a month, six months, and a year.

The principal tides of the semi-diurnal group are (*a*) the lunar semi-diurnal tide at intervals of 12 hours $25\frac{1}{4}$ minutes caused by an ideal moon moving on the equator; (*b*) the solar semi-diurnal tide at intervals of exactly 12 hours, due to an ideal sun also moving along the equator; and (*c*) the lunar elliptic tide, occurring at intervals of 12 hours $39\frac{1}{2}$ minutes, and representing the principal effect of the elliptic motion of the moon round the earth.

In the diurnal group the three principal tides have periods of 25 hours $49\frac{1}{8}$ minutes, 23 hours 56 minutes, and 24 hours 4 minutes.

Now, the assembling and co-ordination of the various data for so complex a series of calculations is very laborious, and occupies a great deal of time. It is possible, however, to arrive at the desired result mechanically, in a much less troublesome manner, by means of an apparatus known as a tide predicter, devised by Lord Kelvin, and described by him in a paper read before the Institution of Civil Engineers in 1880.[2] The principle of the mechanism will be understood from the model shown in fig. 43. The following description is taken from the paper :—

"For each tidal constituent to be taken into account, the machine has a shaft, with an overhanging crank, which carries a pulley pivoted on a parallel axis adjustable to a greater or less distance from the shaft's axis, according to the greater or less range of the particular tidal constituent for the different ports for which the machine is to be used. The several shafts,

[1] It may be interesting to mention that in astronomical work the perturbations or disturbances of the moon and planets are similarly analysed, or broken up into a series of waves, each wave being a regular sine curve with constant period, and each acting independently of the other. This method is known as *Fourier Analysis*, and can be applied to all disturbances of a periodic character.

[2] "The Tide Gauge, Tidal Harmonic Analyser, and Tide Predicter," *Min. Proc. Inst. C.E.*, vol. lxv. The author is aware of the outspoken, and even sharp, contention in regard to the share of Mr. Edward Roberts in the design and production of this machine, as recorded in the discussion on the paper. In justice to Mr. Roberts, he alludes to his claim, but feels that this is not the place for entering into the merits of the case. Mr. Roberts communicated to the Royal Society in 1879 a description of the Tide Predicter constructed under his supervision for the Indian Government.

with their axes all parallel, are geared together so that their periods are to a sufficient degree of approximation proportional to the periods of the tidal constituents. The crank on each shaft can be turned round on the shaft and clamped in any position; thus it is set to the proper position for the epoch of the particular tide which it is to produce. The axes of the several shafts are horizontal, and their vertical planes are at successive distances one from another, each equal to the diameter of one of the pulleys (the diameters of these being equal). The shafts are in two rows, an upper and a lower, and the grooves of the pulleys are all in one plane perpendicular to their axes. Suppose now, the axes of the pulleys to be set each at zero distance from the axis of its shaft, and let a fine wire or chain, with one end hanging down and carrying a weight, pass alternately over and under the pulleys in order, and vertically upwards or downwards (according as the number

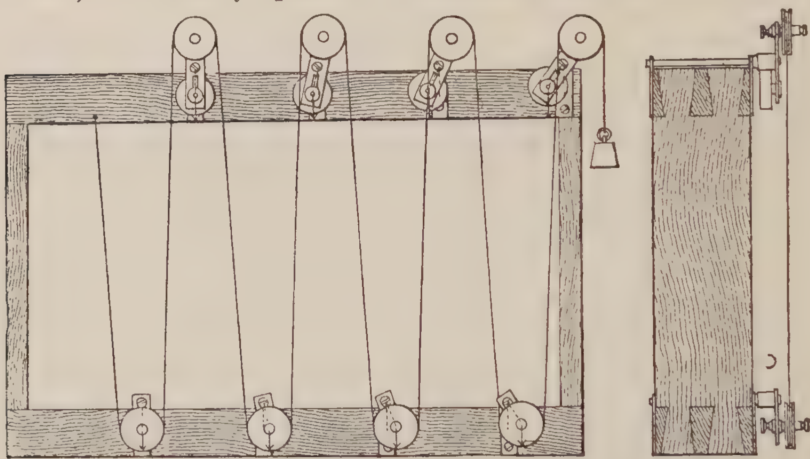


Fig. 43.—Model of Tide Predicter Mechanism.

of pulleys is even or odd) from the last pulley to a fixed point. The weight is to be properly guided for vertical motion by a geometrical slide. Turn the machine now, and the wire will remain undisturbed, with all its free parts vertical, and the hanging weight unmoved. But now set the axis of any one of the pulleys to a distance $\frac{1}{2}$ T from its shaft's axis, and turn the machine. If the distance of this pulley from the two on each side of it in the other row is a considerable multiple of $\frac{1}{2}$ T, the hanging weight will now (if the machine is turned uniformly) move up and down with a simple harmonic motion of amplitude (or semi-range) equal to T, in the period of its shaft. If, next, a second pulley is displaced to a distance $\frac{1}{2}$ T′, a third to a distance $\frac{1}{2}$ T″, and so on, the hanging weight will now perform a complex harmonic motion equal to the sum of the several harmonic motions, each in its proper period, which would be produced separately by the displacements $\frac{1}{2}$ T, $\frac{1}{2}$ T′, $\frac{1}{2}$ T″. Thus, if the machine was made on a large scale, with T, T′ . . . equal respectively to the actual semi-ranges of the several constituent tides,

and if it is turned round slowly (by clockwork, for example), so that each shaft goes once round in the actual period of the tide which it represents, the hanging weight would rise and fall exactly with the water level as affected by the whole tidal action. This, of course, could be of no use, and is only suggested by way of illustration. The actual machine is made of such magnitude that it can be set to give a motion to the hanging weight equal to the actual motion of the water level reduced to any convenient scale; and provided the whole range does not exceed about 30 centimetres, the geometrical error due to the deviation from perfect parallelism in the successive free parts of the wire is not so great as to be practically objectionable."

A machine of this type with some modifications and improvements was constructed for the Indian Government, and every year a revision is made of the tidal constants for 37 ports in the Indian Ocean. These tide tables have been described as "amongst the most admirable in the world." [1]

By similar means the tides are calculated for other ports in various parts of the world. Those for the Port of London are supplied by Mr. E. Roberts, F.R.A.S., and the following analysis of the results during a twelvemonth is an interesting testimony to the general accuracy of the apparatus.

Of a total of 668 high waters observed, during the year ended 31st March, 1915, at London Bridge, 252 were found to be within five minutes of the predicted time and 352 within 6 inches of the predicted height. A further 308 were within 15 minutes of the tabular time and 261 within 18 inches of the tabular height. This accounts for between 80 and 90 per cent. of the predictions, and, having regard to the uncertainties of wind and weather, both of which affect the tides and are, of course, impossible to forecast, it represents an exceedingly close approximation.

Tidal Range.—The highest tides in the world occur at the head of the Bay of Fundy, in New Brunswick, where, at times, a rise of 100 feet is recorded. On the British coast the highest tides occur in the Bristol Channel, culminating in a height of 60 feet occasionally at Chepstow. Inland and land-locked seas, such as the Mediterranean, the Gulf of Mexico, etc., are characterised by very feeble and, in some cases, almost inappreciable tides, and they are accordingly designated tideless seas. The rise in the Baltic is hardly perceptible, but in the narrow Strait of Cattegat, it sometimes reaches 1 foot. Similarly, in the Mediterranean, there are but a few inches between high and low water, while in the Gulf of Venice 2 feet is registered. The tidal range is often affected, as already stated, by conditions of wind and barometric pressure.

The following is a table of approximate tidal ranges at some of the more important ports and maritime stations in both hemispheres. More complete and extended information will be found in the Admiralty Tide Tables, from which the figures are mainly extracted :—[2]

[1] Darwin on *The Tides*, 3rd edition, p. 342.

[2] "Tide Tables for Standard Ports in the United Kingdom and other Parts of the World." Published annually.

Tidal Rises at Certain Harbours.

	Rise in Feet.			Rise in Feet.	
	Springs.	Neaps.		Springs.	Neaps.
United Kingdom—			*United Kingdom—Con.*		
Aberdeen,	12¾	9¼	Grimsby,	19¼	15¼
Aberdovey,	14¼	10	Guernsey, St. Peter Port,	26	18¾
Aberystwith,	14¼	10	Hartlepool,	16	12¾
Alderney,	20	15½	Harwich,	12¾	10½
Alloa,	17½	15	Holyhead,	16	12½
Arbroath,	14	11	Hull,	20¾	16¼
Ardrossan,	10	7½	Immingham,	18¾	15
Avonmouth,	41¾	31½	Inverness,	14	9
Ayr,	8¾	7¼	Ipswich,	13½	..
Ayre Point (I.O.M.),	20	16	Jersey, St. Helier,	34½	26
Banff,	10½	8	Kinsale,	11½	9
Bantry Harbour,	10¼	7¾	Kirkwall,	8½	6½
Barnstaple,	10½	..	Lancaster,	8½	2
Barrow-in-Furness,	28¾	22	Leith,	17¾	14¼
Barry,	37½	28½	Lerwick,	5¾	4½
Belfast,	9½	8	Limerick,	18¾	14½
Berwick-on-Tweed,	15	11½	Liverpool,	29	23¼
Blyth,	15	11	London Bridge,	22¼	18¼
Boston,	22	15	Londonderry,	7¾	5¾
Bridgewater,	18	..	Lowestoft,	6½	5¼
Bridport,	11¼	7¾	Maryport,	25	19
Brighton,	19¾	16	Middlesbrough,	17	12¼
Bristol,	33	23	Milford Haven,	21¾	16½
Buckie,	12	9	Montrose,	14	11
Burntisland,	17¾	14¼	Morecambe,	27	21
Campbeltown,	8¾	6	Needles (I.O.W.),	7½	5
Cardiff,	36½	27	Newcastle,	15	11¾
Cardigan,	12	9	Newhaven,	19¼	14
Carnarvon,	15¾	12	Newport (Mon.),	38	29
Chatham,	18	14¾	Oban,	11¼	8¼
Chester,	10	..	Padstow,	23	16½
Coleraine,	6¼	4	Penzance,	16¼	12¼
Cork,	12¾	10	Perth,	9½	..
Cowes,	12½	9½	Peterhead,	11½	9¼
Dartmouth,	14¼	10½	Plymouth,	15½	12
Deal,	16	12½	Poole,	6½	4¾
Douglas (I.O.M.),	22¼	17½	Portland,	6¾	4¼
Dover,	18	14¼	Portsmouth,	12¾	10¼
Drogheda,	11¾	9	Port Talbot,	29	18¼
Dublin,	13	11	Preston,	17	10
Dundee,	14½	11½	Queenstown,	12	9½
Falmouth,	17¾	14¼	Ramsgate,	15	12
Fishguard,	13½	8¾	Rosyth,	18¾	15
Fleetwood,	27	20½	Ryde,	13½	10
Folkestone,	20	16½	Seaham,	14½	10½
Fowey,	15	11¾	Sheerness,	17¼	14¼
Fraserburgh,	11	8½	Sligo,	11½	8½
Galway,	15½	11½	Southampton,	13	9½
Glasgow,	13¼	10½	Stornoway,	13½	9½
Goole,	13	..	Stromness,	10¼	7½
Gravesend,	19¾	16	Sunderland,	14½	11
Greenock,	10¼	8¼			

TIDAL RISES AT CERTAIN HARBOURS—(*Continued*).

	Rise in Feet.	
	Springs.	Neaps.
United Kingdom—Con.		
Swansea,	27¼	20¼
Tay, Mouth of,	16	13½
Tees, Mouth of,	15¼	12
Thurso,	13¼	9½
Tilbury,	19¾	16
Torbay,	13½	10
Tynemouth,	15¼	10¾
Waterford,	13	10½
Westport,	12¾	9½
Wexford,	5	3½
Weymouth,	7	5
Whitby,	15	11½
Whitehaven,	25¾	18¼
Wick,	10	7½
Wicklow,	9	6½
Workington,	25¾	20
Yarmouth,	6	4½
Youghal,	12½	9½
France—		
Bordeaux,	15¾	12¼
Boulogne,	26	20¾
Brest,	19¾	14¾
Calais,	21	17½
Cherbourg,	17¾	13
Dieppe,	27½	21
Dunkirk,	16¾	13½
Havre,	22	17¾
Rochefort,	16¾	13
St. Malo,	35½	26
St. Nazaire,	16	12
Holland and Belgium—		
Antwerp,	16¾	14½
Flushing,	13	10
Nieuport,	16	13
Ostend,	15	12
Rotterdam,	4¾	4
Texel,	4¼	3½
Ymuiden,	5½	5
Germany—		
Altona,	7	..
Bremerhaven,	13	12
Cuxhaven,	11½	10
Emden,	11½	10¼
Hamburg,	8	7½
Norway—		
Bergen,	4	..
Christiania,	1½	..
Stavanger,	4½	..

	Rise in Feet.	
	Springs.	Neaps.
Mediterranean—		
Alexandria,	1	¼
Corinth,	¾	..
Gibraltar,	3¾	3
Trieste,	2¼	1¼
Tripoli,	2	..
Tunis,	3	..
Valetta,	1	..
Spain and Portugal—		
Cadiz,	12¾	8½
Corunna,	13	11
Lisbon,	12	9
Oporto,	10	8
Seville,	7½	6¼
Africa—		
Aden,	7	4½
Fernando Po,	7	..
Lagos,	2	..
Mozambique,	12	..
Natal,	6½	3¾
Perim,	7	5¾
Sierra Leone,	12	8
Suez,	7	4
Table Bay,	5	3½
Tangier,	8¼	5
Zanzibar,	15	10
Asia—		
Bombay,	14¼	11¼
Calcutta,	17	12
Colombo,	2	..
Madras,	3½	2¼
Hong-Kong,	6½	5
Rangoon,	19	14
America—		
Boston,	10¼	8¾
Callao,	4	..
Galveston,	1	..
Mobile,	1	..
New York,	5	4
Panama,	16	12¼
Philadelphia,	5½	5
Quebec,	18	13¼
San Francisco,	6	5
Valparaiso,	5	..
Australasia—		
Auckland,	9¾	8½
Brisbane,	7½	..
Melbourne,	2¾	..
Nelson,	12	9
Sydney,	5¼	4

Note.—The rise of the tide, as given in the foregoing table, is the difference in level between high water and the datum of the chart. Datums vary considerably with different

nations. In British waters, where the diurnal inequality is small, the Admiralty datum is the level of mean low-water springs; where the diurnal inequality is considerable, it is the level of Indian spring low water. In Denmark, Norway, and Japan the datum is low-water springs; in Holland and the United States, it is mean low water; in Germany, it is about 1 foot below mean low-water springs; in France and Spain, it is lowest possible low water.

Rise, therefore, differs from Range, which is the difference in height between consecutive high water and low water. The divergency between rise and range is most marked in the case of neaps, the low-water level of which is often as much above the level of low-water springs as the high water is below high-water springs. In other words, the tides oscillate more or less equally above and below a central datum line, which may be taken roughly at half the spring rise. The table affords the data from which an approximate idea of the range may be obtained.

A diagram of tidal ranges at Liverpool is given in fig. 15, page 29; the datum is exceptional and purely local, and is taken from the sill of an old dock which has long since ceased to exist; the level of the sill, however, has been carefully preserved.

Exceptional Tidal Phenomena.—It has been stated that usually there are two occasions of high water within a period of 24 to 25 hours, but at Southampton there are four such occasions, in pairs, separated by a slight interval, and due to the passage of the tidal wave round both sides of the Isle of Wight. The same phenomenon occurs at Rochefort, in the vicinity of the Island of Oleron. At Portsmouth there are two sets of three tidal peaks: two explicable as before, and the third attributable to the flux and reflux of the tide in and out of the coastal inlet known as Langston Harbour.[1] At Havre high water remains stationary for a full hour, and only fluctuates to the extent of 13 inches in a period of three hours. In the Gulf of Tonkin and at Cat Island, in the Gulf of Mexico, as also at Pensacola, there is only one tide per diem. These, and other anomalies, elsewhere, have some local explanation which is not in all cases readily apparent.

[1] The English South Coast, generally, is characterised by extra tides, and the Admiralty "Tide Tables" has the following note on the phenomenon:—"Over a considerable length of coast between Portland and Selsea Bill a double tide is experienced; the first high water occurring more or less in consonance with the progression of the tide from the west, the second with an apparently counter tidal undulation or influx of water from the eastward, the result being that near the eastern limit of this section a prolonged rise of tide is caused, which, in the Solent, develops into two distinct high waters with an interval of from one to two hours between them, and this interval increases progressively along the shore westward of the Needles to three and four hours, until, as Weymouth is approached, the double tide corresponding more closely with the time of low water becomes in fact a double low water, and is locally known as the 'Gulder.'"

CHAPTER IV.

SURVEYING, MARINE AND SUBMARINE.

Harbour Surveys—The Base Line—Methods of Measurement—Sextant—Adjustments—Box Sextant—Sounding Sextant—Station Pointer—Section Lines—Soundings—Appliances: Line, Chain, and Pole—Sutcliffe's Apparatus—Alignment—Tide Gauges—Current Observations—Floats—Location—Silt Observations—Diving Operations—Diving-Bells—Dress and Equipment—Rate of Ascent after Immersion.

Marine Surveying.—While generally and appropriately considered as constituting a special department of maritime work, with a field and purview of its own, marine surveying has, at the same time, certain of its operations so closely associated with the ordinary routine of harbour engineering that some reference to them, if not actually imperative, is at least very desirable.

It is not contemplated, however, to enter into a detailed explanation of the principles underlying the carrying out of a complete hydrographical survey, nor, in fact, to do more than describe in brief outline the main operations involved in the preparation of a chart of a portion of the coast line for the purposes of harbour design. The methods of more extensive surveys and the elucidation of particular problems should be sought in geodetic text-books specially written for the purpose.

The operations immediately concerning the engineer in the design and construction, as also in the maintenance work of a harbour, are (*a*) the making of a local survey, (*b*) the taking of soundings, and (*c*) the determination of the direction and velocity of tidal or fluvial currents. Our observations, therefore, will be directed to these points.

Harbour Surveying.—Even if reliable coastal charts are at hand, it will still generally be necessary for the engineer himself to make a local survey covering some particular area, and he would proceed to do this by devising, first of all, a suitable and convenient scheme of triangulation. The preliminary step is that of laying down and measuring with extreme accuracy a **base line,** which shall be sufficiently long and so completely accessible as to enable the whole of the desired area to be plotted therefrom, for which purpose its extremities should command a view of as many salient points as possible. The length required for a base line will naturally depend upon the extent of the survey, but as some rough indication it may be said that 1,000 yards should be ample for an ordinary harbour survey, while, in many cases, much shorter lines will suffice. Wherever available, a fairly level site should be chosen adjacent to the shore line, with an uninterrupted outlook. The line can then be laid down and measured by means of a surveyor's chain,

or a steel tape, taking care to check the measurement several times and to compare the chain or tape with some standard length. With the steel tape, the tension should be maintained constant, say at a pull of 10 or 15 lbs., as defined by a spring balance. The temperature should be noted and adjustment made, as wide variations will appreciably affect the readings.

Direct measurement of this kind is the only method of any really practical value for engineering purposes, but under certain circumstances in which absolute accuracy is not essential, other and less trustworthy means may be resorted to. Thus, where no suitable site is at hand, or when a base line, instead of lying along the shore, has to be carried over the water, the method of computation from the angular observation of a ship's mast may be employed.

In such a case, a vessel is moored in some position, as A (fig. 44), which commands a full view of the line, B C, to be measured. All three angles of the triangle A B C are taken simultaneously by separate observers at the points A, B, and C, and, in addition, the observers at B and C take the "mast-head angle" of the vessel at A, from which the height of the mast being known, the lengths B A, C A can be computed as the bases of vertical right-angled triangles; thus B A = height of mast × cotangent of angle. With the sides B A, C A of the triangle A B C so obtained, and the horizontal angles already observed, the length B C can be determined.[1]

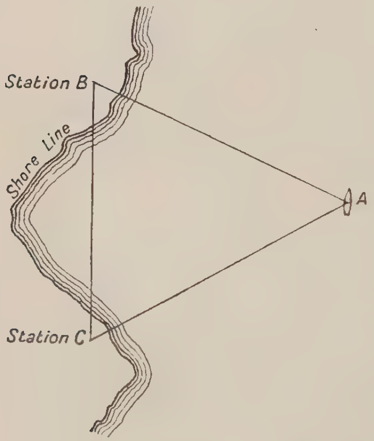


Fig. 44.—Measurement of Base Line by means of Masthead Angle.

There is still another method of indirect measurement employed, but it is only serviceable in very rough approximations, and, as it admits of a considerable degree of error, it should only be applied to base lines of very considerable length—4 or 5 miles or over. It is based on the speed at which sound travels, and, as this varies with the temperature, it is very essential to ascertain the thermometric reading at the time of observation. Sound travels at the rate of 1,090 feet per second at freezing point (0° C. or 32° F.), and practically 2 feet per second faster for every additional degree Centigrade or $1\frac{1}{7}$ feet per second for every degree Fahrenheit. Variations in the barometer do not affect the result, but allowance must, of course, be made for the

[1] The case assumed is that in which the observer's eye is approximately level either with the foot or the summit of the mast. For other positions, neither sighting line would form an exact right angle with the mast, and a more complex trigonometrical determination would be necessary for accurate work.

velocity of any wind which may be blowing, and as this is difficult to estimate with any precision, measurements of distance by sound are best taken on a calm day. The method of taking observations is as follows :—

A gun or small mortar is fired from the station at one end of the line, while at the other end the beats of a watch are counted between the appearance of the flash and the arrival of the sound. For this purpose chronometer watches with five beats to two seconds are usually employed. A number of readings should be made and the mean of these taken. As each beat represents two-fifths of 1,100 feet—say 440 feet—it is obvious that the distance computed may be in error to the extent of 220 feet either way.

Extension of the Base Line.—Having defined the primary, or base line, which will generally be short in relation to the full range of the survey, it next becomes necessary to extend it by successive well-conditioned triangles [1] to conspicuous points on each side until the maximum line requiring to be plotted has been obtained. Thus, in fig. 45, the lines C D and E F are progressively obtained from the base A B. From the stations A and B angular measurements are taken to C and D, and those points being defined, angular measurements are taken from them to E and F.

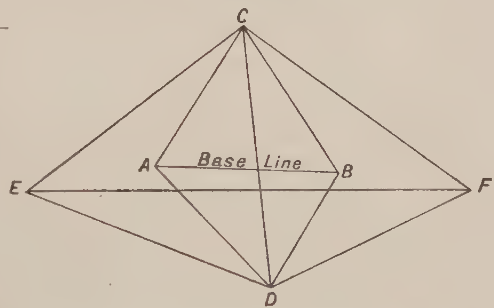


Fig. 45.—Method of extending the Base Line.

With the principal lines of the survey laid down, the next step is to fill in the coast line. For this purpose the observer follows the shore, on foot wherever practicable, otherwise in a small boat, and fixes various positions by angular observation to main or secondary stations. It is then necessary for him to sketch in the configuration of the coast between these positions, and this is done as he proceeds. If practicable, straight lines may be run between several points and the offsets measured, as in a land survey. In addition to the coast line, the surveyor fixes the position of prominent features of the adjacent landscape, and outlines the topography of the ground. Prominent objects form useful landmarks for determining bearings.

Sextant.—Although for angular measurement on land the theodolite is more frequently employed, the chief instrument of the harbour surveyor is the sextant (figs. 47 and 49), which combines most of the functions of the theodolite, with the advantage that it does not require a fixed support, and can be equally well manipulated on a surface in motion. It is thus specially

[1] Triangles equiangular, or nearly so—that is, with angles of about 60° and never less than 30°. Such angles are essential for accurate plotting.

adapted to the conditions of working from a boat. It can also be used to take simultaneous observations of two moving bodies.[1]

The principle of the sextant is the arrangement of two mirrors in such a way that the angular distance between two objects can be determined by their apparent coincidence when one of them is seen by direct vision and the other by reflection. Thus in fig. 46 M_1 and M_2 are two mirrors, the lower half only of M_2 being silvered; X and Y are the two objects, and C is the eye of the observer. Y is sighted by direct vision through the upper unsilvered half of M_2, X, which should, if possible, be the brighter of the two objects, is seen by double reflection at M_1 and M_2. Since the angle of incidence is equal to the angle of reflection, as $\alpha\ \alpha$, $\beta\ \beta$, and the angle between the perpendiculars to the mirrors is equal to the angle between the mirrors, we have, by Euclid I. 32:—

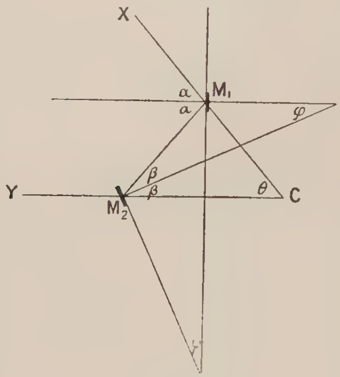


Fig. 46.

$$\theta = 2\alpha - 2\beta = 2(\alpha - \beta)$$

but likewise, $\varphi = \alpha - \beta$

therefore $\theta = 2\varphi$

i.e., the angle θ, subtended by the two objects at the point C, is measured by twice the angle φ, between the mirrors. The sextant was originally constructed for an arc of 60° (hence its name), but now has a range up to 75°, and accordingly measures angles up to 150°. It is not advisable, however, to attempt readings in excess of 140°. The scale is so graduated that the angles may be read directly without doubling. The arc is usually subdivided to 10 minutes, and a vernier gives readings to 10 seconds. Angular distances exceeding the range of the sextant can be obtained in two stages by sighting to some intermediate object.

In making observations, the sextant is held so that its plane lies in that of the objects to be observed. The telescope[2] is directed towards the left-hand or lower object, until it is seen through the unsilvered portion of the horizon glass (M_2). The index arm, or radius bar, should then be moved until the second object appears by reflection from the index glass (M_1) in the silvered portion of the horizon glass and the two objects apparently coincide.

[1] The sextant takes actual angular measurements—*i.e.*, not necessarily angles in azimuth, such as are requisite for plans. It is, therefore, when used in this connection, only suitable for taking angles over a fairly level surface, unless the necessary corrections be made for difference in altitude.

[2] In practical work the sextant is often used without the telescope. It is much easier to work a sextant in a boat with the unaided eye, and the results are quite satisfactory.

The following are the principal **adjustments of the sextant** :—

1. The index glass (M_1) must be perpendicular to the plane of the arc. To test this, bring the index arm to a central position, and, placing the eye near the mirror and nearly in the plane of the arc, observe if the portion of the arc reflected in the mirror appears in continuation of that seen directly. If not, adjust by means of the screw provided.

2. The horizon glass (M_2) must also be perpendicular to the plane of the arc. Sight some well-defined object with the horizon glass, and, bringing the index arm in the vicinity of zero reading, note whether, in moving the index arm a little backwards and forwards, the reflected image coincides completely with the direct image. If not, make the necessary screw adjustment.

3. The two mirrors should be parallel when the index is at zero. This is checked by bringing the direct and reflected images of a single object into coincidence on the horizon glass. The angular reading should then be zero. If this is not the case, the actual reading, or "index error," as it is called, should be allowed for in subsequent readings, or the index glass may be adjusted by a key.

4. The centre of the axis of the index arm and the centre of the arc must be coincident. This and faulty graduation must be reckoned errors of construction. They are not easy to ascertain exactly, and can best be manifested, if they exist, by reference to some establishment, such as Kew Observatory, or the National Physical Laboratory, where apparatus for their detection exists.

5. In instruments fitted with sighting wires there is an adjustment in connection with the line of collimation of the telescope, which should be parallel to the plane of the arc. Surveying sextants do not usually possess sighting wires, but when these are present, the tube containing them should be revolved until they are both parallel to the plane of the arc. Then sight two distant objects on one wire, and see if their images are still in contact on the other wire when the instrument is moved to the required extent. Any correction necessary would be made by means of the screws in the telescope collar.

The use of the sextant is chiefly associated with navigation and the taking of such astronomical observations as are essential thereto. In its ordinary form, therefore, it is constructed with this object in view. For the harbour engineer there are two modifications of design, which are better adapted to the particular work he has to perform. These are the box sextant and the sounding sextant.

The **Box Sextant** (fig. 47),[1] as the name implies, is a sextant with the working parts under cover and, therefore, protected from accidental injury. For an instrument in constant use and not, as in the case of the nautical sextant, requisitioned for one or two diurnal observations only, this is a very desirable precaution. The instrument, moreover, is thereby rendered com-

[1] Taken by permission from *Surveying and Levelling Instruments*, by W. F. Stanley.

pact and conveniently portable. The principle of its construction will be readily understood from figs. 47 and 48. The first is a perspective view of the box sextant ready for use; the second shows the working parts under the face. A (fig. 47) is the axis of the index glass I (fig. 48) carrying a vernier which reads into the arc scale to single minutes, the arc itself being divided to half degrees. The index glass, fixed on a toothed segment, is worked

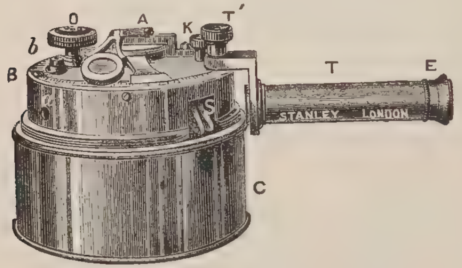


Fig. 47.—Perspective View of Box Sextant.

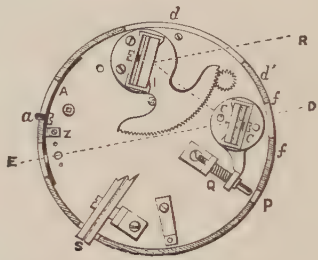


Fig. 48.—Box Sextant under the Face.

by the pinion, moved by the milled head O, about one and a half turns of which cause the index to traverse the entire scale of 120°. The reflected ray, coming from R, enters at a wide window in the side of the box, and passes to E along the path indicated by the dotted line; the direct ray from D is received through the smaller window *f f*. The horizon glass has vertical adjustment screws *c c*, and the differential adjustment with the index glass is made by the screw at P acting against the spring Q. The position of the eye is marked by the pinhole at E. T is a detachable telescope secured to the sextant by means of the milled head screw T′. For close work the telescope is not used. Shades, discs of parallel glass on arms, rise from the back of the face at pressure on the ribs at S.

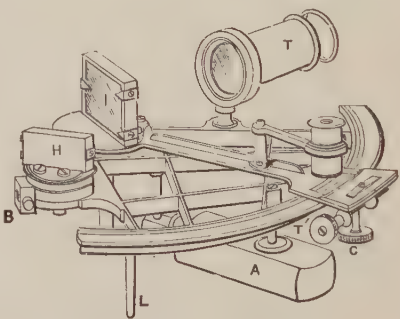


Fig. 49.—Sounding Sextant.

The **Sounding Sextant** (fig. 49)[1] follows more closely the construction of the ordinary sextant, but is made more solidly and has a greater optical range. The index glass (I) is large, about $2\frac{1}{4}$ inches by $1\frac{1}{4}$ inches, while the horizon glass (H) is small, $1\frac{1}{2}$ inches $\times$ $\frac{3}{4}$ inch, and has its surface entirely silvered—*i.e.*, without any plain portion, as in the ordinary sextant. The eye receives the direct ray from one of the two objects, the angular measurement of which is to be taken, above the reflected image of the other object with the narrow frame of the horizon glass between. For terrestrial obser-

[1] Taken by permission from *Surveying and Levelling Instruments*, by W. F. Stanley.

vations this does not appreciably affect the perception of apparent contact.

Bearing of the Base Line.—An essential operation in a survey is the determination of the direction and bearing of the base line in reference to the meridian, and this is done by observing the horizontal angle made by the line with the sun at some moment when the true bearing of the sun is known. Thus, if the horizontal angle is α°, the sun lying to the right, and the sun's true bearing β° in a north-easterly direction, the true bearing of the base line is $(\alpha - \beta)^\circ$.

Plotting the Survey.—Having completed the field work, the survey will be plotted by any of the recognised methods in such cases, the angles being laid down either with the aid of a table of chords,[1] or from the less important stations by means of a protractor. Generally speaking, these methods will suffice, but there is an instrument for dealing with the determination of a point by angular readings to three other fixed points, of which some notice is desirable, as it is peculiarly associated with marine survey work. It is known as the **Station Pointer** or **Chorograph.** The instrument has three long, flat arms or splayed straight-edges, all radiating from a common centre: two of them are movable, the middle one is fixed. A graduated circular arc attached to this middle arm, with vernier indices on the side arms, enables the instrument to be accurately adjusted to any given combination of angles. This performed, the instrument is laid upon the plan so that the straight-edges pass through each of the three fixed stations. The centre of the instrument then marks the desired point.

Section Lines.—On the completion of the survey the next step is to obtain information as to the levels and contours of the under-water portion, and this is done by means of soundings taken along certain lines, the positions of which are known and can be plotted from fixed points along the shore. The distance apart of these lines will be determined by circumstances, and will also be governed by the consideration stated in the footnote on p. 79. The lines are generally taken at right angles to the coast or, in the case of a river, to the axis of the channel, and it is convenient to arrange them, as far as practicable, parallel to one another so as to ensure covering the area uniformly.

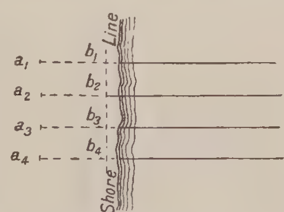


Fig. 50.—Parallel Section Lines from Shore.

Where there is an island or detached reef, or in the vicinity of a shoal, it is very desirable to take soundings along a series of radiating lines, in order that the obstacle may be completely outlined and its contours accurately determined.

For sections of moderate length, straight lines may be projected forward with the aid of two poles, or upright stakes, carrying flags, one at the water's

[1] Computed from the formula: Chord = 2 radius × sin ½ angle.

edge and the other some 50 to 80 feet back, but, in any case, advisably not less than a fourth of the line to be run. As shown in fig. 50, sighting poles would be set up successively at $a_1\, b_1$, a_2, b_2, etc. For greater lengths a prominent object at some distance should be picked out, and utilising this as a rear sight, lines may be run radially from a series of points along the shore,

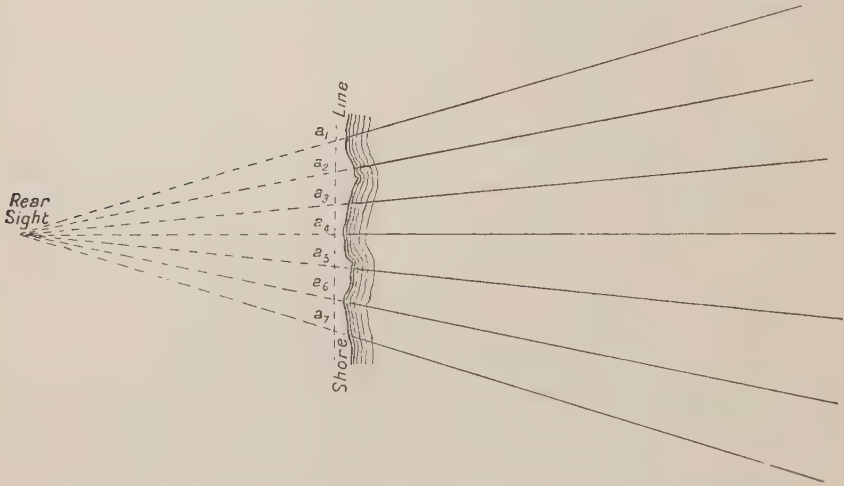


Fig. 51.—Radiating Section Lines from Prominent Landmark in Rear.

as in fig. 51, until they become unduly oblique, when a fresh rear sight should be found. The positions of these rear sights should, of course, be accurately defined on the chart or survey.

In river work two shore lines may be run, one on each bank, as in fig. 52, and these may be fixed relatively to each other by angular measurement to their extremities, or they may be run radially to a rear sight, as in fig. 51.

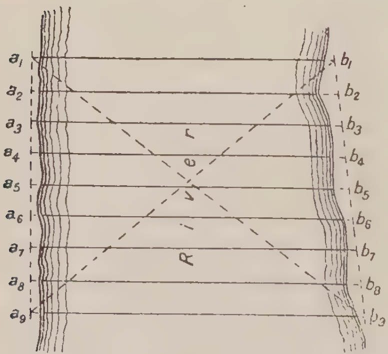


Fig. 52.—Parallel Section Lines across River.

Soundings.—The taking of soundings is a very common operation in navigation, but the appliances used in that connection and the methods in vogue are by no means identical with those characteristic of harbour practice. In the latter sphere much greater precision and accuracy are essential than can be afforded by the somewhat rough and ready appliances often employed in connection with shipping.

Manifestly the simplest way of taking a sounding—that is, of ascertaining the depth of water at any spot—is to lower a pole or weighted line until the bottom is reached. If the pole or line be graduated to linear measure, the

depth can be read directly therefrom. This method is perfectly satisfactory when performed from a stationary base, as, for instance, when the operator is standing on a quay wall or on a boat which is moored, and in this way it is, of course, only applicable to single dips.

When it is desired to take a series of dips along a given line in any direction, the base, if afloat, must obviously be movable ; and while a boat, no doubt, may be alternately moved and moored so as to fulfil the condition stated above, yet the process would be slow and tedious. It is evidently preferable to adopt some method of taking soundings in close and uninterrupted sequence while the boat is in continuous motion.

The lowering of a line under these circumstances would lead to inaccurate readings, as, by the time the bottom was reached, the travel of the boat would have produced considerable inclination in the line, so that it would be no longer vertical. Seamen get over this difficulty by throwing or heaving the lead, which weights the line, some distance ahead of the boat, giving it time to reach the bottom before the boat passes perpendicularly over it. The reading is then taken at the moment of verticality, as near as can be judged. This procedure, however, involves a considerable loss of time in raising the lead to the surface for each throw. Dips, therefore, cannot be taken closely together.

In the hands of an expert operator working from a row-boat in comparatively shallow water, it is possible for the lead to be maintained a short distance above the bottom between the dips while a series of readings are taken. Inclining his body in the direction in which the boat is travelling, the operator will hold the line suspended from the hand nearest the bow until a dip has to be made. Then, dropping the lead, passing the line quickly into the other hand, and simultaneously erecting his body, he will be able to take a reading at the instant when the line passes through the vertical position.

The method is somewhat rough and crude, but, except in deep water, it gives fairly reliable results. The hand-lead, ordinarily used for the purpose, has a length of line not exceeding 30 fathoms, which is more than ample for any engineering requirements. The length of line for purposes of harbour soundings need hardly exceed 50 or 60 feet and very often much shorter lengths than this will serve.

The **sounding line,** if of hemp or wire, is graduated by tags of different texture, shape, and colour, so as to be identifiable by night as well as by day (fig. 53). Strips of woollen material, cotton, leather, serge, and even string are utilised to give variation. If a chain line be used, copper-stamped links are sometimes substituted for the tags (fig. 54), but the contact of two metals of different electrical potentiality sets up electrolytic action and leads to corrosion. An excellent sounding line is a $\frac{3}{16}$-inch phosphor-bronze wire cable, the wire being marked by a swivel at each odd number of fathoms commencing at 3, with intermediate marks on any convenient system. A

chain, or wire, in fact, though more difficult to handle with wet hands,[1] is preferable to a hemp line on account of the excessive shrinkage of the latter, amounting, when new, to as much as 5 per cent. during the course of a day's work. New lines should, in fact, be avoided for this reason, and all lines should be well wetted just before use. Finally, they should be tested frequently—before and after each line of soundings, if possible—by some standard length, which, for all practical purposes, may be marked on the boat itself.

A seaman's lead is usually an octagonal bar, weighing from 8 to 10 lbs., and such a bar, with a cross-section of about 2 square inches, is best adapted for rapid sinking. For a very soft bottom, a flat disc weighing about 5 lbs. is sometimes found more suitable, as there is less tendency for a lead of this shape to sink into mud or other soft material. If, however, it be desired to penetrate through the mud to firm ground, a bar or ball must be employed.

Fig. 53.—Wire Sounding-line.

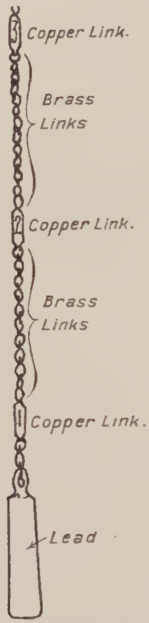


Fig. 54.—Chain Sounding-line.

For soundings in shallow water, in depths, say, not exceeding 20 or 25 feet, a **pole** is frequently used. As regards convenience in handling, white pine, which is light, forms the best material from which to make it. The pole is either circular in section, or oblong, about 2 inches by 3 inches, with hollowed faces, painted and graduated in feet and quarter-feet; sometimes in feet and inches. It is generally shod with a flat-bottomed shoe, the weight of which should be just sufficient to assist in sinking the pole to the bottom, and no more.

The drawback attaching to all these appliances—line, chain, and pole—is their liability to miss some prominent protuberance in the bottom due to the isolated nature of the dips and the distance which lies between them. Except in perfectly still water, it is necessary to keep the boat moving at a certain rate in order to steer it and prevent it from being deflected out of its course by currents. It is difficult, therefore, under ordinary circumstances, to take soundings with the lead line at intervals of less than 10 feet. And a good deal may lie hidden in 10 feet. As a matter of fact, soundings are

[1] The leadsman usually wears a pair of woollen mittens while manipulating the dips.

To face p. 79.]

Fig. 55.—Sutcliffe's Sounding Machine.

frequently taken at much greater distances.[1] Furthermore, there is the effect of wave motion, which interferes very materially with the accuracy of the readings.

Fro these reasons, and for others which it is unnecessary to enumerate, a more complete and reliable system of recording depths in connection with detailed surveys in harbour work is highly desirable, and a number of attempts have been made to supply apparatus which will conform with the requirements of the case. Chemical and electrical agencies have been proposed, and tested with varying, but generally unsatisfactory, results. They are too sensitive in action and too delicate in adjustment for use in exposed situations amid unstable surroundings. Whatever possibilities they may contain, at anyrate they have not yet been put into a working form, and mechanical appliances still seem to supply the only practicable means of dealing with the problem.

For detail sounding work in estuaries, harbours, and coastal inlets, where there is some degree of shelter, the most serviceable and efficient machine with which the author is acquainted is one designed and patented by Mr. Fielden Sutcliffe, of Liverpool. Having had occasion to use it many times the author feels in a position to speak confidently on its merits and capabilities. The following is a description of the machine :—

Sutcliffe's Sounding Apparatus.—The apparatus illustrated in figs. 55 and 56 consists of three parts: the Sounding Machine (shown fitted up on a boat), the Horizontal Distance Measurer, and the Section Plotter.

"The *Sounding Machine* consists of a wheel with a grooved rim, on which is wound a fine steel wire having the lead attached to its free end. The wheel is mounted in a frame which is arranged for clamping to the gunwale of a boat at its starboard quarter. The sounding-boat is also equipped with a sprit and leading block over the starboard bow.

"On the back of the wheel there is a spiral reel, on which a second line, called the 'Preventer Line,' is wound. The free end of this line is passed through the sprit block and attached to the lead, and it, together with the sounding line and a given length of base, forms a right-angled triangle, the length of base being the distance from the sprit block to the sounding line. The function of the Preventer Line is to restrain the lead from trailing astern, and thereby to obviate the necessity of casting the lead forward for each

[1] It is largely a question of the scale to which the soundings plan is to be plotted. Within a square inch of paper it is only practicable to record, at the most, one hundred observations which shall be distinctly legible. If the scale then be 88 feet to the inch, soundings cannot be recorded more closely than 8·8 or 9 feet, and if 208 feet to the inch not more closely than about 21 feet. As many large harbours are charted to smaller scales than these, it is obvious that for plotting purposes it is useless to take readings too near together. On the other hand, for smaller areas and for detail work, especially in connection with the location of detached obstacles and the clearance of a fairway for shipping, frequent and close readings are essential, and these will be plotted to a correspondingly larger scale.

dip, to admit of the lead being carried near the bottom, so that frequent dips may be speedily taken and that intermediate obstructions (should they occur) may not be passed without notice, and to insure the verticality of each dip of the lead. The wheel and the reel are so proportioned relatively to each other and to the horizontal distance from the sounding line to the sprit block, that they each pay out or take in the requisite amount of their respective lines to maintain the sounding line vertical at all depths of its range.

"The wheel measures 10 feet in circumference at the bottom of its rim groove; consequently, the length of sounding line paid out per revolution is 10 feet, with fractions in proportion.

"At the front of the wheel frame there is a scale, along which a pointer is caused to travel at a rate proportional to the vertical travel of the lead, to indicate the depth below the surface of the water. Indications are also afforded by a pointer at a graduated ring on the face of the wheel.

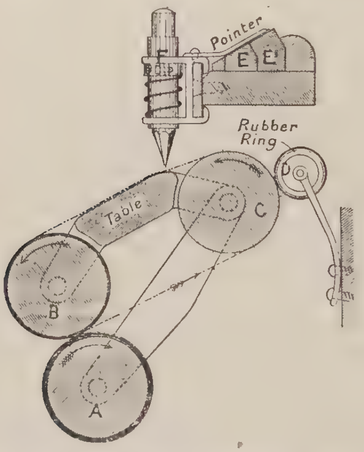


Fig. 56.—Mechanism of Section Plotter in Sutcliffe's Apparatus.

A. Paying-out roller. D. Guide wheel.
B. Winding-on roller. E E′. Scales.
C. Guide roller. F. Pencil bracket.

"In taking soundings with this machine, the operator first sets the lead at the surface of the water and the pointer at zero.

"He then grasps the rim of the wheel with his left hand, releases a catch with his right hand, and allows the wheel to revolve until the lead strikes the bottom; then, reversing the motion of the wheel, he reads at the pointers the depth indicated at the instant when he feels the sounding line become taut. Continuing the movement a little further, he raises the lead clear of the bottom in readiness for the next dip. The operator is assisted in the picking up of the slack line by the reaction of a coil spring on the wheel axle.

"The *Horizontal Distance Measurer* consists of a drum on which is wound a length of fine steel wire or steel cord. The drum is mounted, with its axis vertical, in a bracket fixed at the stern of the sounding-boat. As the boat moves away from its starting-point, to which one end of the wire is attached, the drum rotates and pays out the wire, the unwinding being regulated by a hand-wheel and screw acting upon a brake band. The revolutions of the drum are made to indicate on a counter the length of wire paid out, or to act in conjunction with the Section Plotter.

"The *Section Plotter* is a device for recording on paper a line of soundings to scale simultaneously as the sounding proceeds, and requiring only the

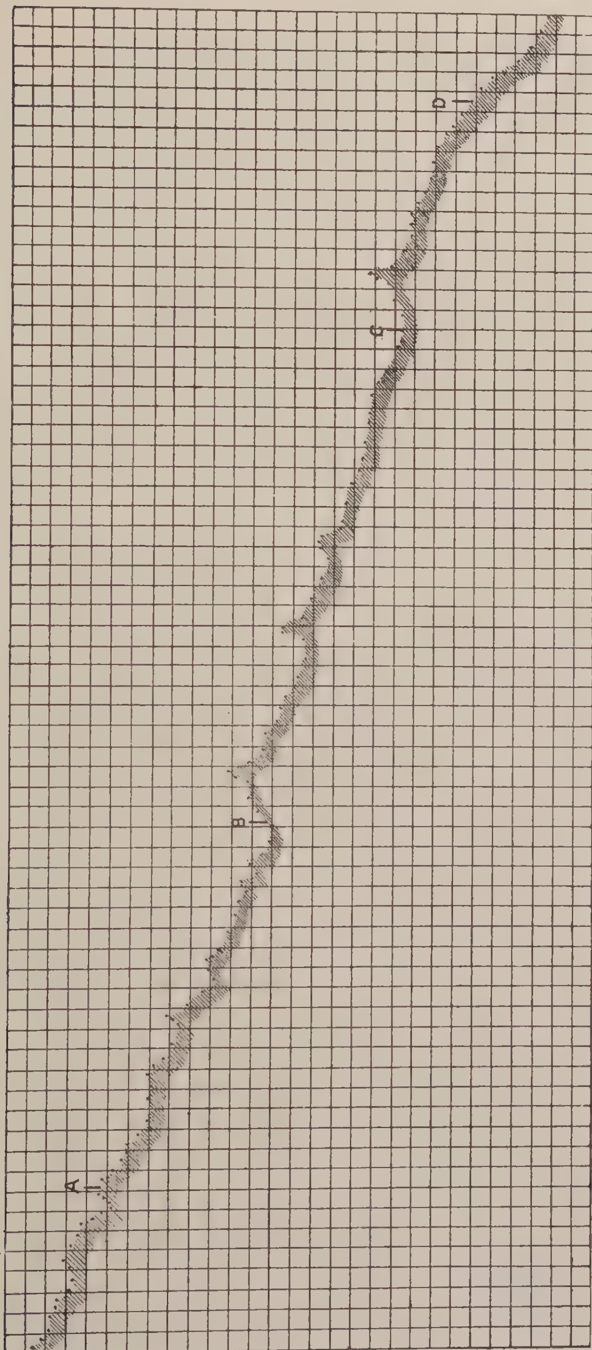


Fig. 57.—Line of Soundings taken by Sutcliffe's Apparatus. The dots only were indicated by the machine; the etching added subsequently.
A, B, C, and D are cross sights to landmarks or other fixed points.

addition of a datum line to make a complete section. By its means a band of paper is made to traverse at right angles the track of the movable pointer of the sounding machine at a rate proportionate to the paying out of the distance measuring line. Thus the pointer, the motion of which across the paper represents the amount of rise and fall of the lead, acquires relatively to the paper a second movement corresponding to the horizontal travel of the lead. The pointer carries a marker, sprung just clear of the paper, and the operator, on taking a sounding, has only to tap the marker with a finger of his right hand in order to record the sounding on the paper, by means of a dot which denotes the position of the lead on the section plane, both vertically and horizontally.

"It is convenient to use tracing paper on the plotter and to have the vertical scale $\frac{1}{4}$ inch per foot and the horizontal scale $\frac{1}{10}$ inch per foot, so that, on laying a section upon a sheet of paper ruled to $\frac{1}{4}$ inch squares, soundings may be read at sight and quantities above any given line may be readily computed.

"For positions where a distance measuring line is unsuited, the plotter may be actuated by clockwork, but, in that case, the section produced is not to scale horizontally, and distances must be fixed by cross-sights or bearings."

In the example (fig. 57) of a line of soundings taken and plotted by Sutcliffe's Apparatus, the letters A, B, C, D represent the position of cross sights to landmarks, by means of which the operator verifies his distances.

During the process of taking soundings in a tideway, two points demand the constant attention of the operator. One is the *alignment of the dips*, and the other is the *mutation of the water level.*

Alignment is difficult to maintain in a cross current, and the boat needs to be carefully watched to see that it does not drift out of its course. If a line of soundings be taken from the shore or quay out towards the open, a couple of poles or other suitable uprights, one at the water's edge and the other some distance back, as explained on p. 76, should afford adequate guidance. When the bank is steep, it may be necessary to give the rear pole greater elevation than the other in order to be able to range them both with the eye from a lower level. If a line of soundings lie between two fixed points, such as stakes fixed into opposite banks of a river, the distance between them not being very great, a rope may be stretched taut from one to the other. In this case, by providing the rope with tags at regular intervals, the exact distance of each sounding can be recorded. When there is only one fixed point, distances may be read off a cord or rope, similarly tagged, and paid out from the boat as it proceeds. Failing this, the position of the boat at each dip must be fixed by angular measurement from the shore, with the aid of the sextant or theodolite.

Thus an observer stationed at one end (A) of the shore line A B (fig. 58) reads in succession the angles B A C, B A D, etc., which enable the positions of the dips to be plotted. Alternatively readings may be taken from the boat

of the angles A C B, A D B, etc. In either case the positions of the points A and B and the direction of the line B K must be clearly defined beforehand.

Variations in the Water Level should be noted at regular and stated intervals (say every five or ten minutes) by an observer stationed at a tide-

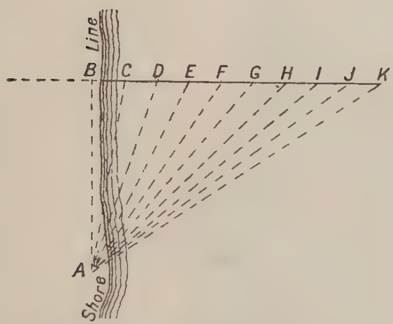


Fig. 58.—Fixing Position of Soundings on Section Line by Angular Measurement.

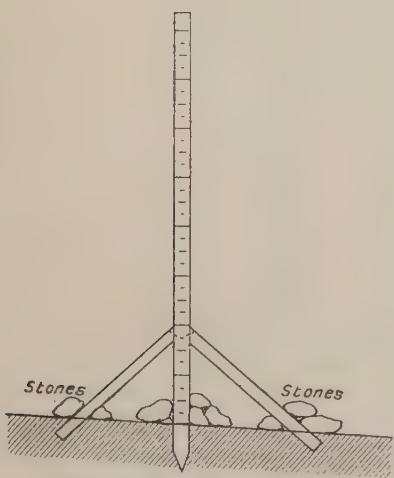


Fig. 59.—Temporary Tide-gauge on Beach.

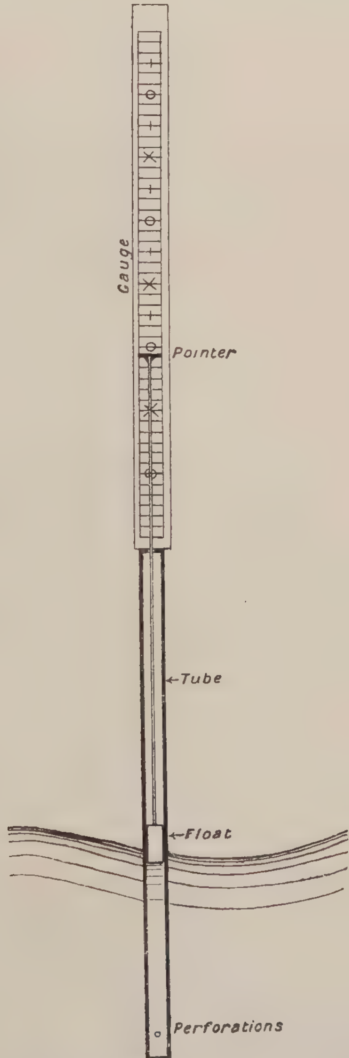


Fig. 60.—Tide-gauge for use in rough or choppy Water.

gauge adjacent to the site of operations. The operator in the boat also notes the dips which correspond to the same intervals of time, and, by subsequent comparison with the tide-gauge readings, the proper correction can be made

by which all the soundings are referred to one datum line, either local and temporary, or established and general.

A **Tide-gauge** is an appliance for the purpose of indicating changes in the sea surface level. In its simplest form it consists of an upright stake or post (fig. 59) driven into the shore or bank, and graduated to linear measure. In some situations a single post may suffice to indicate the whole tidal range;

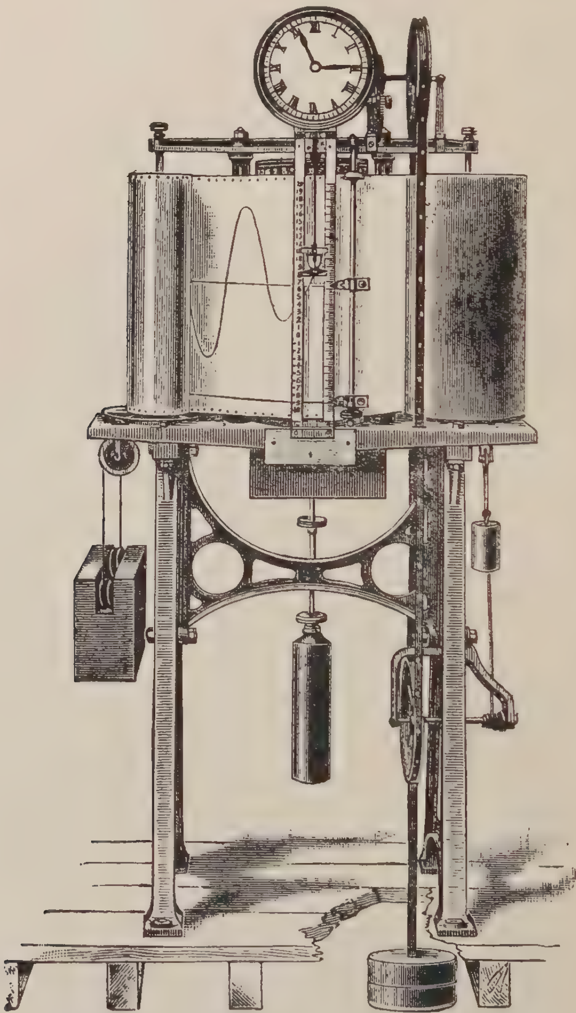


Fig. 61.—Automatic Tide-gauge Recording Machine.

in other cases a number of posts may be necessary, extending across a sloping shore from high water level to low water level and forming a series of steps.

In cases where there is any swell, the gauge may consist of a rod or indicator, with a float at its lower end inclosed in a tube, the bottom of which

is perforated as shown in fig. 60. Such an apparatus may be affixed to a quay or other vertical wall.

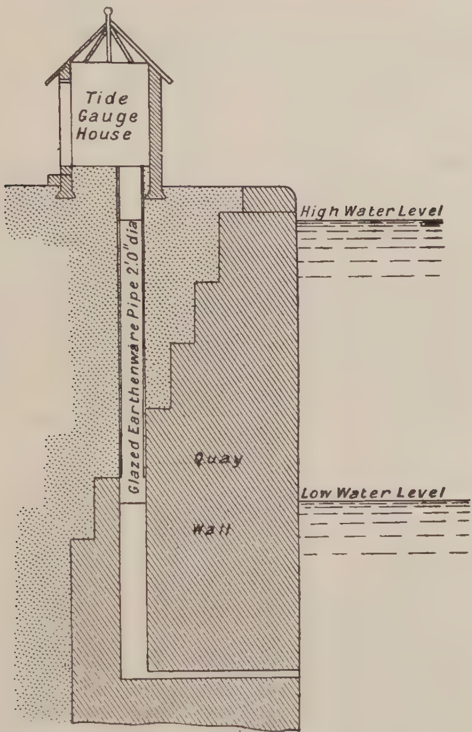


Fig. 62.—Tide-gauge House on Quay Wall.

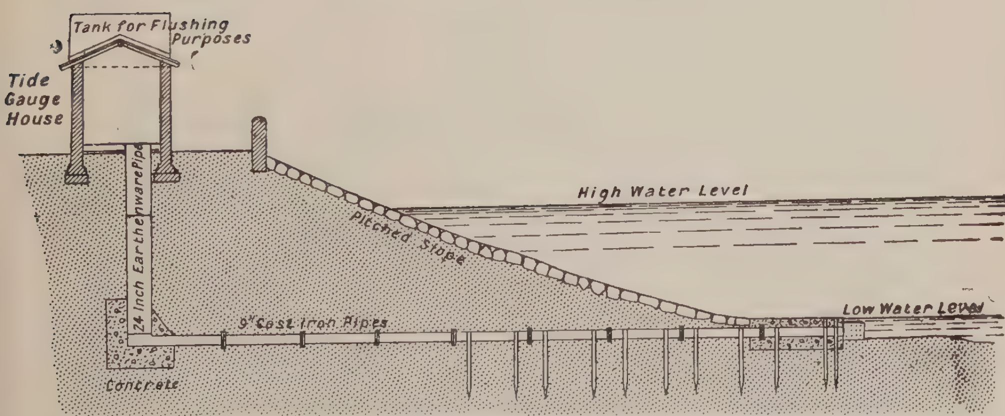


Fig. 63.—Tide-gauge House on River Bank.

In important localities it is customary for tide-gauges to be more elabor ately constructed and to possess self-registering apparatus. A well or

tube in free communication with the sea is fitted with a float supporting a graduated upright stem, which passes upwards to a scale and pointer. For self-registering purposes the float is connected by means of a chain or cord with a movable pencil situated so as to mark the surface of a paper-covered cylinder, which is rotated by clockwork.

Such an automatic tide-gauge recording machine is shown in fig. 61. The pencil moves vertically up and down under the influence of the float seen through the opening in the floor, and leaves a continuous curved trace on the roll of paper which is wound off one drum on to another, the movement being actuated and controlled by clockwork. Suitable reduction gear is introduced to confine the vertical action of the pencil within convenient limits. In the case illustrated the paper is 16 inches high, divided to scale to give a range of 30 feet. The clockwork similarly revolves the drum to a fixed scale of time, as indicated by the marginal dots, through which lines may be drawn. The machine illustrated, manufactured by Messrs. A. Légé & Co., of London, works for a fortnight without rewinding, and the roll of paper lasts for eighteen months.

In other types a single drum or cylinder is made to suffice. The edges of the paper are carefully trimmed and joined together, so as to present an even surface, which has, of course, to be cut on removal. In this case the record curves intersect each other; but, as the tide of each day is later than that of the preceding day, no difficulty is experienced in identifying the successive records, provided the period is not allowed to exceed a semi-lunation—*i.e.*, a fortnight.

In taking soundings, not only is it necessary to have a tide-gauge immediately adjacent to the scene of operations, but when the scope of these is extensive, several gauges at various points will be required, because fluctuations in the water level are frequently local, and they are by no means uniform.

Examples of permanent tide-gauge stations are shown in figs. 62 and 63: one situated at the edge of a quay wall, and the other on a river bank. In muddy places a flushing tank is provided for cleansing the gauge well and maintaining free access of water to it.

The Determination of Currents.—It is important to the engineer to know the directions taken by tidal currents at various times during the day, and to observe their relation in regard to the configuration of the coast-line and the maintenance of navigable inlets. To acquire this knowledge, he has in many instances to fall back upon personal observation, and one of the earliest steps in connection with the laying out of harbour work will be to acquire the requisite data in regard to current flow.

Floats.—The obvious method of observing the set or direction of a current is by means of some floating object. Any substance drifting upon the surface of the water would appear to afford a means of recognising the trend of tide or stream. It must be pointed out, however, that the matter is not so easily

disposed of, and that there are some complexities behind an apparently simple experiment.

In the first place, paradoxical as it may seem, the topmost layer of the water may flow in a different direction to the lower layers or main body. Fresh water has a less specific gravity than salt water, and the two do not readily mix. A fresh-water stream, encountering a tidal inset, will, therefore, flow uppermost for some distance before becoming incorporated. The wind also is capable of exerting so powerful an influence on the surface of a body of water in motion as sometimes to make the uppermost layer move directly counter to the under portion. Then again, there may be cross-channel currents producing a spiral action, in such a manner that the particles of

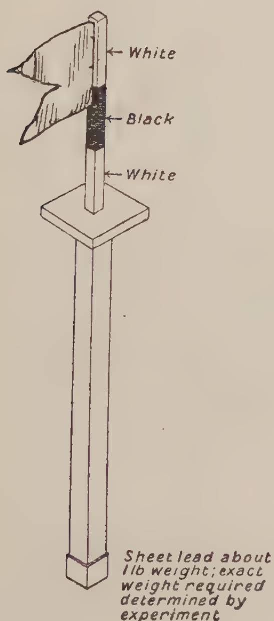


Fig. 64.—Float.

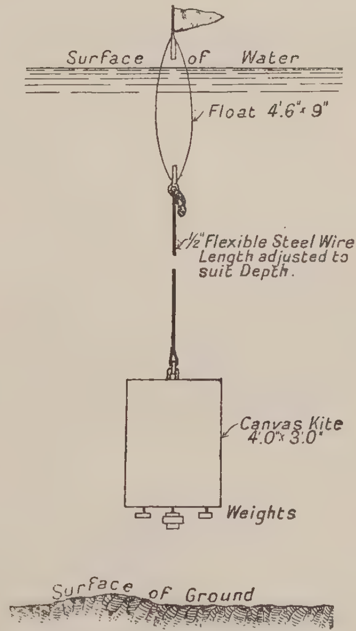


Fig. 65.—Float for Deep Water.

water move across the bottom in a direction totally different from that which they take along the surface.

And not only is direction affected, but velocity is very much more involved. The velocity of a moving stream varies considerably throughout its depth. All that can be said is that the maximum lies somewhere below the surface and above the bottom, and is generally near the upper limit. There are sources of retardation, due not only to aerial movement, but also to friction with the ground.

This being so, floats for determining the flow of currents should extend some depth into the water so as to partake of the influence of as many layers

as possible. They must also project sufficiently above the surface level to enable accurate observations of their positions to be taken, without, however, presenting too much surface to the action of the wind.

A circular or square pole, with a wooden cylinder or prism at its lower end, weighted so as to float vertically, affords a suitable form of instrument. Such an indicator is shown in fig. 64.

On the Mersey, one form of combined current "drag" and float employed consisted of a skeleton box frame covered with canvas, the width of each side being 3 feet and the depth 4 feet (fig. 65). The drag was weighted with detachable weights and suspended from the float by means of a steel wire the length of which could be adjusted to suit the depth. The float was of wood, conical in shape, with apices top and bottom, $4\frac{1}{2}$ feet long and 9 inches central diameter. At the top was a socket to receive a small flag. The drag floated with a few inches of cone showing above the surface of the water. It was found that the drag drifted a little faster than the float, with the result that there was some slight retardation of the former due to the pull of the float, but the error was considered so small as to be negligible. Observations were taken every fifteen minutes from a boat accompanying the float, a light anchor being dropped two minutes beforehand, so that the boat swung alongside the float without actually touching it. After reading the angles and taking a sounding, the anchor was pulled up and the course resumed.

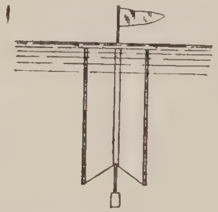


Fig. 66.—Surface Float.

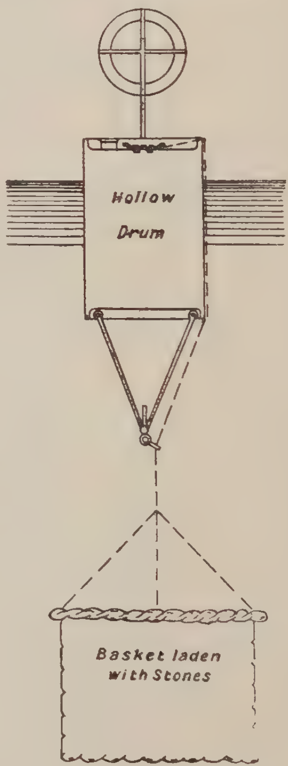


Fig. 67.—Float used on River Avon.

A float, used simultaneously for ascertaining the surface velocity, consisted of a light deal frame 3 feet deep, cruciform in section, with a flat top 9 inches square, on which was mounted a small flag (fig. 66). The frame was weighted at the underside to an extent just sufficient to bring the flat top level with the surface of the water, or rather to just submerge it, so that any slight breeze might have no effect upon it. It was generally found

practicable to take the readings for both indicators from a single boat at intervals of 15 to 20 minutes.[1]

On the River Avon an empty 5-gallon oil-drum has been used, sunk to almost complete immersion and ballasted by a basket of stone attached below it in the manner shown in fig. 67. On top of the drum is fixed a sighting mark consisting either of a semaphore, flag, or disc.

For purposes to which no great accuracy is essential, any convenient buoyant object, such as an empty keg or barrel, may serve to indicate the direction and general speed of the current, and in positions where a small object can be easily seen, an angler's float will do admirably. Small pieces of cork or wood suggest themselves as equally utilisable. An orange makes a good float under convenient circumstances of visibility. Its specific gravity is very little less than that of water; hence it floats nearly wholly immersed, exposing little or no surface to atmospheric action.

Current observations may also be made by anchoring a boat and streaming a patent log, but this method, while affording some approximation of speed, gives no adequate indication of the variable direction of the current.

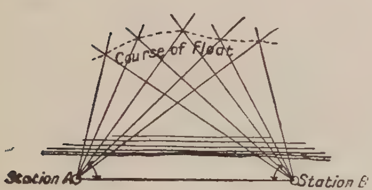


Fig. 68.

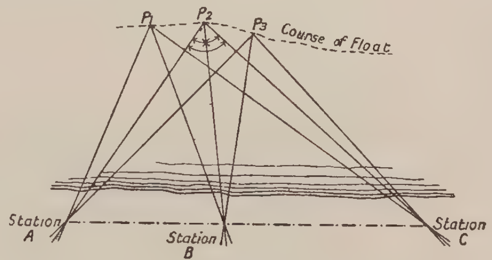


Fig. 69.

There are two ways of taking the necessary observations, which **fix the location of the float** at any desired instant of time.

In the first method, two operators, each with theodolite or sextant, are stationed at a fixed distance apart along a base line on the shore (fig. 68). At concerted signals each operator measures the angle subtended by the line joining the other operator and the float. The intersection of the lines forming these two angles respectively, determines the position of the float at the time of observation.

For the second method (fig. 69) one operator will suffice, but he must generally be provided with two sextants. Following closely in a boat the course of the float, at any assigned moment and as rapidly as possible, he takes the readings of the two angles which the float makes with three fixed objects ranged along the shore frontage, conveniently situated as near as possible abreast of his position and preferably co-linear or nearly so. These angles having been plotted on a piece of tracing paper, the latter may be

[1] Mace on "Harbour Surveying," *Min. Proc. Liverpool Engineering Society*, vol. xxxiii.

adjusted over a plan, or a station pointer (p. 75) may be used, so that the three lines pass through the fixed points on the shore. When this condition is fulfilled—and, except in the instance about to be cited, there is only one position corresponding to any pair of angles—the point may be pricked through.

The method fails if all four points lie on the circumference of a circle (fig. 70). All angles in the same or equal segments of a circle are equal. Thus the angles at P_1 are equal to the angles at P_2. It is better for the middle station (B) to lie nearer the shore line than stations A and C, and this should be the case wherever practicable. The observed angles should also be large. The plotting of small angles is always unsatisfactory, even when the points do not lie on a circle.

For angles less than 90°, the position of the point P may be plotted directly, without recourse to tracing paper or station pointer, as follows:—Let the observed angles A P B and B P C be $x°$ and $y°$ respectively. In fig. 71 make

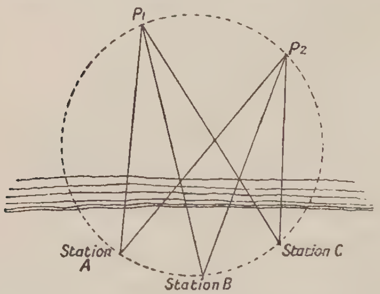


Fig. 70.

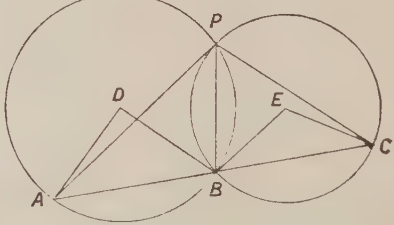


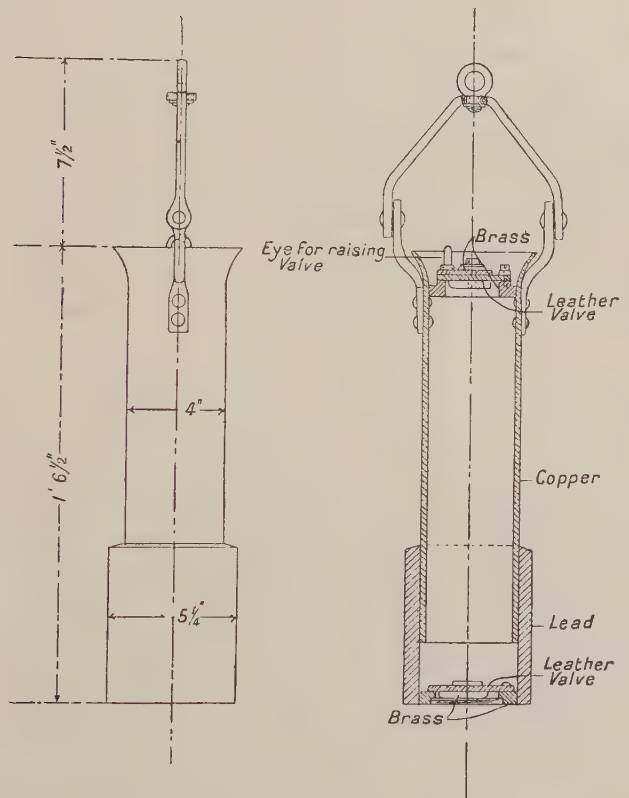
Fig. 71.

the angles B A D and A B D each equal to $90° - x°$. With D as centre describe the circle A B P. Similarly make the angles C B E and B C E each equal to $90° - y°$, and describe the circle B C P. The point P falls at the intersection of the two circles. The proof of the construction lies in the fact that the angle at the centre of a circle is double the angle at the circumference (Eu. III. 20). It will be observed that the best condition for determining the point P is when the circles intersect at right angles.

It has been remarked that for this method of locating the position of the float, two sighting instruments are generally necessary. It is manifest that the observations must be as simultaneous as possible. Any hurry in reading the first angle prior to adjustment for the second would lead to error. It is preferable, therefore, to fix both angles and defer the readings of either until that has been done. As a check, the angle containing the two subsidiary angles may also be read, but this involves the provision of a third sextant, with, probably, the aid of another operator.

Sediment Sampling.—Among the data required in connection with the planning of harbour works in tidal districts, or wherever the water is charged with sand and silt, is some estimation of the quantity of **sediment in suspension**

at different depths, at successive stages of the tide and under varying conditions of wind and weather. The simplest means of obtaining samples is by lowering a tube with valves at each end, which can be closed at the desired depth. A specimen tube, known as a silt trap, used in the Mersey Estuary, is shown in fig. 72.[1] It is made of copper, some 18 inches long and 4 inches



Elevation. Section.

Fig. 72.—Silt Trap.

diameter, weighted on the underside and fitted top and bottom with upward-acting non-return flap valves. The tube is suspended by a lead line marked to give the necessary readings of depth. On lowering the tube rapidly, the valves are kept open by the pressure of the water rising in the tube until the descent of the tube is arrested at the required depth, when they close automatically. The tube is then withdrawn with the sample enclosed, and on reaching the surface, this is poured through the upper valve into any graduated receptacle, where it is allowed to settle and the sediment is measured.

[1] *Vide* Mace on "Harbour Engineering," *Min. Proc. Liverpool Engineering Society*, vol. xxxiii.

A number of samples are, of course, essential in order to arrive at a reliable result, and these should be taken periodically at regular intervals.

Where the sediment is mud of a nature too impalpable for volumetric determination, it may be estimated by weight in the following manner :—A piece of chemical filter paper should be carefully dried, so as to be entirely free from moisture, and then weighed. Placing this in a funnel, the sample should be slowly passed through, taking care that the sediment is kept in suspension until the whole is decanted. The filter paper is then thoroughly dried and the weight again taken. The increase in weight over the former weighing represents, of course, the weight of the sediment. Very delicate apparatus and accurate manipulation are required in these operations.

Diving.—Perhaps the most interesting, not to say romantic, feature of harbour engineering work is the use of diving apparatus in connection with the preparation of submarine foundations. A very large proportion of the operations necessary to the satisfactory stability of breakwaters and quays has to be conducted under water, and much of it would be almost impracticable without the aid of diving-bells and diving-dresses. Natives of the East Indian Seas engaged in pearl fisheries do, it is true, remain under water for appreciable periods without any special apparatus for the supply of air, but the strain is very great, causing bleeding at the nose, mouth, and ears, and if unduly prolonged, leading to fatal results. Apart from these physical effects upon the agents engaged, there is much interruption to the operations, which, in the case of structural work, would be inimical to its satisfactory accomplishment, and the haste with which the operations have to be performed would be incompatible with the exercise of care and accuracy. A regular and constant supply of atmospheric air to workers below the surface enables them to remain on duty for some time without any serious discomfort, and for this reason alone the use of air chambers and diving suits has become an integral accompaniment of all maritime operations.

The **Diving-Bell** (fig. 73) is a metallic chamber of sufficient capacity to accommodate any number of workers, from one man up to a dozen or more. The chamber, which is formed of mild steel plates, carefully rivetted together and caulked so as to be absolutely water-tight, is suspended from the jib of a crane or overhead traveller, or from any lifting appliance afloat or ashore, and so raised and lowered as the case may be. In the interior are seats and foot-rests for the occupants during ascent and descent, and shelves for tools. Signalling gear is provided, and many bells are now fitted with electric light and with telephonic apparatus. In addition to this equipment, there are the necessary air-valves and pipes for maintaining a supply of fresh air at the required pressure. At the top and sides of the bells are observation lenses, affording a view of the environment. The bell is ballasted with cast-iron kentledge, placed in a special chamber, to enable the structure to sink without endangering its equilibrium. The size of bells varies very considerably. Two instances may be quoted as examples. The diving-bells used on

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Fig. 73.—Steel Diving-Bell.

the Dover Harbour Works were 17 feet long by 10½ feet wide by 6½ feet high, inside measurement. At Marseilles the dimensions were 66 feet in length, 22 feet in width, and 6½ feet in height.

Diving-bells may be kept in comparatively free communication with the upper air by means of a cylindrical tube, carried up above the surface of the water and surmounted by a special chamber, known as an air-lock. This chamber has two tightly fitting doors ; one giving on to the open, and the other into the tube. After entrance to the chamber from above or below, the door is closed and the pressure equalised with that of the bell or the atmosphere as the case may be. The air-lock thus forms a convenient means for the transmission of material to and from the base of operations—for the passing out of excavation and the taking in of stone and cement ; and it becomes, in fact, essential, if the progress of the work is to be uninterrupted.

In many cases, however, owing to the bulk and weight of the foregoing apparatus, submarine operations have been mainly, and even entirely, carried on with the aid of individual divers, each equipped with a helmet diving-dress and capable of acting in perfect independence of any submerged chamber. Where the locality is free from currents, there can be little doubt that this method of working is preferable in many ways ; but a current of even 1 knot constitutes a troublesome force for the diver to contend against, and it throws a strain on the air-pipe which is not desirable. It must be borne in mind that a diver, when immersed, is a very buoyant object, and that he necessarily finds it difficult to withstand any appreciable lateral force. It is comparatively easy for him to be swept off his feet, and even a moderate flow makes his foothold far from secure. Within the author's recollection is the case of a diver engaged in dock work at Liverpool who was suddenly swept through a culvert by the careless raising of a penstock in the vicinity. The incident, with its fatal consequence, illustrates the uncertainty of a diver's equilibrium and the great risk he sometimes runs.

Apart from this drawback, it must be admitted that work, as a rule, can be more expeditiously performed by men moving in perfect freedom over a large area, than is possible when they are confined within a narrow space, where there is a limit to the number of men employable and the likelihood of their impeding one another's movements.

Diving-Dresses.—The diver's outfit comprises the helmet, the dress, the air-pipe and life-line, and the air-pump.

The *helmet*, which is spherical in shape, is of highly planished tinned copper, as also is the breastplate or corselet to which it is connected, though gun-metal is sometimes employed for the latter. Connection is effected by means of segmental screw neck-rings of gun-metal, the joint being rendered perfectly water-tight by turning the helmet through an angle of 45°. The brestplate is moulded to the shape of the shoulders on which it sits, sometimes with a padded bearing. It receives the collar of the india-rubber dress over a series of brass screws through corresponding eyelet holes. Gun-

metal flanges and wing nuts form a secure and impervious connection. The headpiece is fitted with side and front lights in the form of round or oval plate glasses, set in brass frames, with stout wire-guards. The front glass is detachable by unscrewing, or hinged to open. Air should be introduced into the helmet in such a way as to pass closely over the surfaces of these glasses, and so prevent the condensation of the diver's breath upon them. The other fittings of a helmet are the inlet and outlet valves of the air supply, the latter of which is equipped with a regulator, so that the driver can control his supply of air to a nicety. The inlet valve is so constructed that air is allowed to enter freely, but cannot possibly escape that way, and, in the event of damage occurring to the supply pipe, by closing the outlet valve, the apparatus would retain sufficient air to enable the diver to return to the surface.

The *dress* is in one complete piece, made of solid sheet india-rubber between double-tanned twill. It is fitted with vulcanised india-rubber cuffs and collar, the former being sufficiently close fitting to the wrists to prevent the entrance of water, and the latter pierced with holes to correspond with the clamping screws of the breastplate. In English practice the number of these holes is about a dozen; in French practice, three. The cuffs have generally to be expanded with metal expanders, shaped like shoe-horns, to admit of the passage of the hands, but, in some cases, a bead is moulded on the edge of the cuff, which enables it to be rolled back over the hands. Should the cuffs not prove sufficiently water-tight, the writer has found it a good plan to bind the wrists with a band of moistened chamois leather before the cuffs are put in place.

As it is no uncommon occurrence for a little water to enter the dress through leakage, or occasionally through allowing the outlet valve to be open rather too widely, the diver, before putting on the dress, removes his outer garments and dons a guernsey, drawers, and stockings, as protection from wet and also as padding to his body. For deep or cold water these habiliments may be doubled or trebled. He wears a pair of canvas socks over the feet of the dress to protect it when walking about without shoes, and, if his work is likely to lead him into rough and rocky places, an outer suit of canvas overalls is desirable.

The *boots* are strapped on at the last moment before descending. They are either of specially stout leather, heavily shot with lead, or cast in brass with leather uppers. Additional weight is generally provided for the body of the dress by loading the breast and back with lead pads slung across the shoulders.

The diver's personal equipment is completed by a leather waist-belt containing a knife in a sheath. India-rubber gauntlets may be added, but in this country most divers work without them.

The *air-pipe* is made of the best india-rubber hose with a core of either hardened steel wire, tinned to prevent rusting, or of brass or copper wire. The pipe may be made to float or sink by adjusting the weight of metal. It

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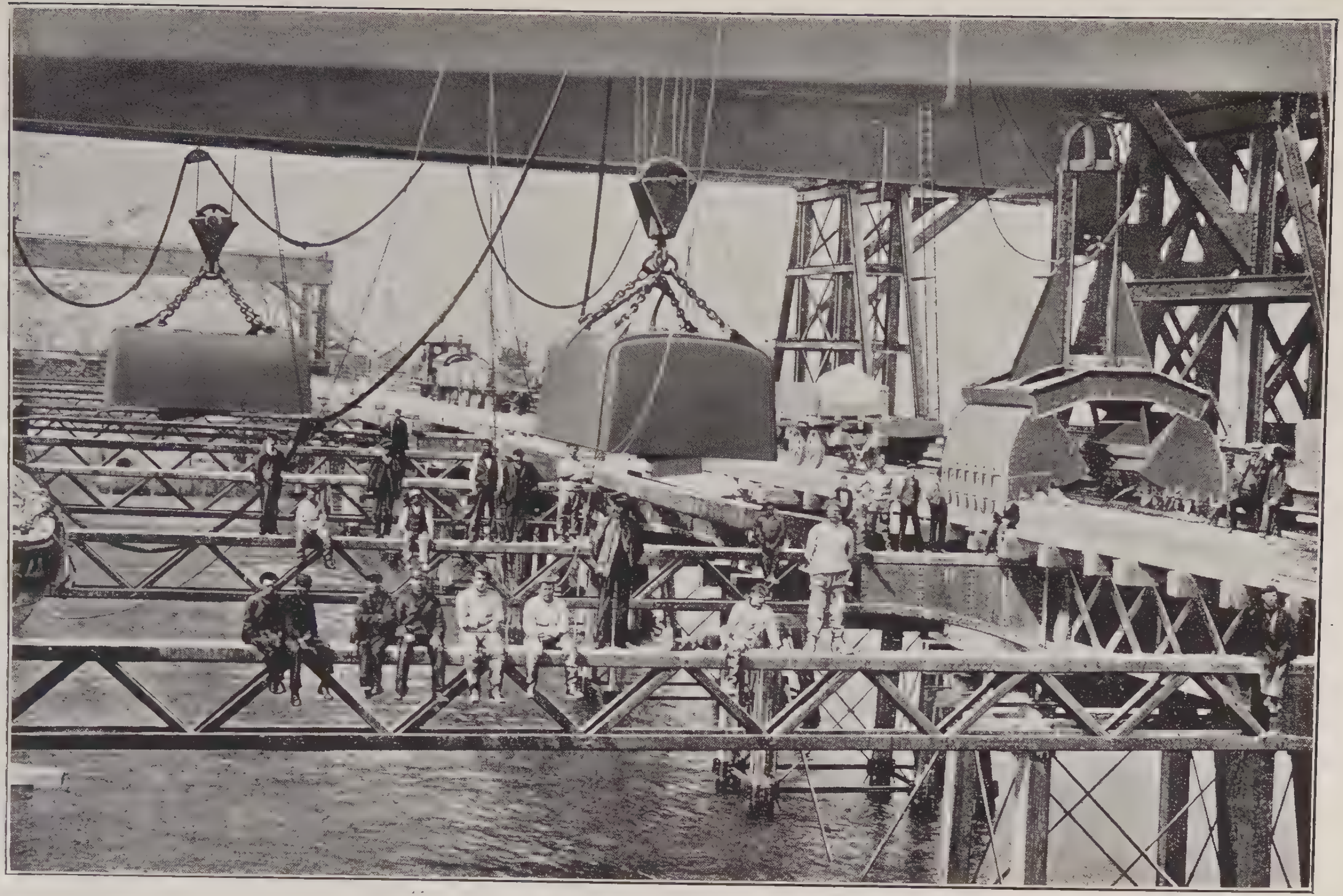


Fig. 74.—Divers and Diving-Bells at Dover Harbour Works.

[*To face p.* 95.

Fig. 75.—Submersion of Diving Bells at Folkestone Harbour Extension Works.

should be tested to a pressure of 200 or 300 lbs. per square inch. After being screwed up to the helmet, the pipe is led and secured under the diver's left arm, so as to be conveniently at his command, and thence it passes upward to the pump. A life-line of stout cord is fastened round the diver's body. Both life-line and air-pipe are paid out together through the hands generally of a single attendant, though they are sometimes in charge of two men. The life-line also acts as a communication cord, according to a code of preconcerted signals, but the most modern outfits are furnished with special speaking-tubes, or with telephonic apparatus, as also with an electric glow-light.

The *pump* is usually double acting, worked by a couple of men, with either single, double, or triple cylinders, according to the depth of water and the pressure required. It is furnished with a gauge indicating both these data.

The qualities needed in a professional diver are not exceptional, but preference will naturally be given to men of nerve and intelligence. The first descent, no doubt, is always more or less a trying experience from its very novelty. The sense of helpless confinement in the midst of a strange and artificial environment, a feeling of oppression, and the increased pulsation all tend to render the initial trip below water (as the writer's experience went) somewhat uncomfortable, if not a source of trepidation. The disagreeable sensations, however, pass away with acclimatisation and practice. Almost anybody in health may make a descent in perfect safety; but for regular and continuous work under water, full-blooded men with short necks are not desirable subjects; neither are those suffering from palpitation or from poor and languid circulation; nor intemperate and generally unhealthy men. Diving is said to be good for the lungs owing to the compressed air affording an increased supply of oxygen and deepening the respiration.

A diver of ordinary powers may descend to a depth of 100 feet with impunity, and may even reach 150 feet without ill effects; but deeper descents are not easily made, and are rarely recorded. The greatest depth to which any diver has descended by authentic testimony is 210 feet, at which point the pressure on his body was 90 lbs. per square inch in excess of atmospheric pressure. Harbour work very seldom entails diving in water exceeding 10 fathoms in depth.

The following table shows approximately the pressures sustained over and above the ordinary atmospheric pressure at varying depths:—

Depth. Feet.	Pressure. Lbs. per sq. in.	Depth. Feet.	Pressure. Lbs. per sq. in.
10	4½	80	35½
20	8¾	100	44½
30	13¼	120	53¼
40	17¾	140	62¼
50	22¼	160	71
60	26½	180	80

Care should be taken in descending and, more particularly, in ascending, not to move too rapidly. After prolonged immersion under pressure the blood becomes saturated with nitrogen, and when the pressure is relaxed the surplus gas is given off. If the process of decompression be carried out too rapidly, bubbles of nitrogen will be formed in the circulatory system (*caisson disease*) with possibly fatal results. The malady is not usually developed after immersion at moderate depths. It is quite safe to come to the surface as speedily as may be desired from a depth of 30 feet or even a little more, but for depths of about 40 feet some precaution is desirable, and above 50 feet is absolutely essential in order to avoid harmful effects. The table on p. 97 shows the degrees of retardation to be observed in rising from depths not exceeding 16 fathoms.

The following regulations usually observed in regard to divers' working hours—except, of course, in cases of a temporary nature or of special urgency—afford an idea of divers' capabilities. For depths not exceeding 60 feet, a shift consists of four hours net, not counting the time taken by the diver to dress. He is allowed a period of fifteen minutes during each shift for rest, and another fifteen minutes at the end for undressing. One or more shifts per day may be worked according to the needs of the case.

The minimum number of attendants required for a single diver is three—one for the signal line and air-pipe, and two to work the pump. For two divers an additional man is required to look after the second signal line and pipe. Pumpers and signalmen may relieve one another at their respective duties.

The following notes on the care of diving apparatus are extracted from Messrs. Siebe, Gorman & Co.'s manual :—

"After the day's work is over, the joints of the air-pipes must be carefully cleaned and the pipes coiled away in the helmet chest. The diving-dress should be cleaned, and, if by any chance wet inside, or if the diver has, perchance, urinated in it, it should be turned inside out and hung up *in the shade* to dry; the dresses, if used in salt water, *should be washed* at least once a week in *clean fresh water*. The underclothing must be kept dry and well aired.

"When in store, the pump and its fittings must be kept clean and free from verdigris, and, if likely to be out of use for some time, it should be occasionally oiled and the handles turned two or three times, in order to prevent the piston leathers getting hard. If the pump has been lying by for a considerable time, then it would be well to have it taken to pieces by a good fitter and examined to see that it is in proper working order. When a piston rod works loose, the screws at the top of the stuffing-box, in the case of the double-acting pumps, should be turned a little with a spanner. Only good olive oil and neat's-foot oil mixed should be used for lubricating.

"Should the diving-dress, from constant use or accident, get leaky, it is easily repaired by laying two or three coats of india-rubber solution on each

Ordinary Time-Limits in Deep Water, Stoppages during Ascent, and Approximate Air-supply needed during Work, in accordance with Professor Haldane's Recommendations, and the Latest British Admiralty Practice.

Depth. Feet.	Depth. Fathoms.	Pressure in Lbs. per Sq. Inch.	Time under Water from Surface to Beginning of Ascent.	Stoppages in Minutes at Different Depths. 30 Feet.	20 Feet.	10 Feet.	Total Time of Ascent in Minutes.	Number of Cylinders Needed.	Revolutions of Pump per Minute.
0 to 36	0 to 6	0 to 16	No limit.	..	..	..	0 to 1	1	15 to 20
36 ,, 42	6 ,, 7	16 ,, 18½	Up to 3 hours.	..	..	..	1 ,, 1½	1	25 ,, 30
			Over 3 hours.	..	..	5	6		
42 ,, 48	7 ,, 8	18½ ,, 21	Up to 1 hour.	..	..	..	1½	1	30
			1 to 3 hours.	..	..	5	6½		
			Over 3 hours.	..	..	10	11½		
48 ,, 54	8 ,, 9	21 ,, 24	Up to ½ hour.	..	..	..	2	2	20
			½ to 1½ hours.	..	..	5	7		
			1½ to 3 hours.	..	..	10	12		
			Over 3 hours.	..	..	20	22		
54 ,, 60	9 ,, 10	24 ,, 26½	Up to 20 minutes.	..	..	..	2	2	25
			20 minutes to ¾ hour.	..	..	5	7		
			¾ hour to 1½ hours.	..	..	10	12		
			1½ to 3 hours.	..	5	15	22		
			Over 3 hours.	..	10	20	32		
60 ,, 66	10 , 11	26½ ,, 29½	Up to 15 minutes.	..	..	..	2	2	25
			¼ to ½ hour.	..	..	5	7		
			½ to 1 hour.	..	3	10	15		
			1 to 2 hours.	..	5	15	22		
			2 to 3 hours.	..	10	20	32		
66 ,, 72	11 ,, 12	29½ ,, 32	Up to 15 minutes.	..	..	2	4	2	25
			¼ to ½ hour.	..	3	5	10		
			½ to 1 hour.	..	5	12	19		
			1 to 2 hours.	..	10	20	32		
72 ,, 78	12 ,, 13	32 ,, 34½	Up to 20 minutes.	..	..	5	7	2	25
			20 to 45 minutes.	..	5	15	22		
			¾ to 1½ hours.	..	10	20	32		
78 ,, 84	13 ,, 14	34½ ,, 37	Up to 20 minutes.	..	..	5	7	2	30
			20 to 45 minutes.	..	5	15	22		
			¾ to 1½ hours.	..	10	20	32		
84 , 90	14 ,, 15	37 ,, 40	Up to 20 minutes.	..	3	5	10	2	30
			20 to 40 minutes.	..	5	15	22		
			40 to 60 minutes.	3	10	15	30		
90 ,, 96	15 ,, 16	40 ,, 42½	Up to 20 minutes.	..	3	5	11	2	30
			20 to 35 minutes.	..	5	15	22		
			35 to 55 minutes.	5	10	15	32		

Note.—The number of cylinders needed is calculated on the supposition that the pump does not leak more than 20 per cent. at pressures up to 60 lbs.

side of the seam, rubbing it with the finger as much as possible and allowing each coat to dry before the next is applied; the sides of the seam may then be laid down, and two or three coats applied in the same manner to the channel of the seam, when the prepared twill (which should have an extra coat laid on and dried) may immediately be applied and well pressed down by the hand. Superfluous solution may be removed with a piece of india-rubber, but it is better to lay it on the proper width so as not to require cleaning off. Diving-dresses should never be packed away in a wet or damp state; they must be thoroughly dried, both inside and out, before so doing, otherwise they will mildew and become so rotten as to be of very little service afterwards. The following represents an easy and efficient mode of drying the diving-dress:—Take two pieces of wood each about 8 feet long, nail or screw them together in the form of a St. Andrew's cross, place them inside the dress, and pass another piece through the arms to keep them distended; the dress can then be set in an upright position until it is dry. In case the diver urinates in the dress, it should be turned inside out, washed with clean water, and then allowed to dry.

"Should the helmet have been lying by for some time, the valves must be unscrewed and examined to see that they are free from verdigris; they may be slightly greased with tallow, and new springs should be fitted if necessary. All the screws of the helmet breastplate should be kept clean, and occasionally wiped with an oily rag."

CHAPTER V.

PILING.

Use of Framework in Maritime Structures—Association of Piling therewith—Varieties—Bearing Piles—Sheeting Piles—Materials for Piles—Timber—Varieties—Destructibility and Preservation—Metal—Concrete—Reinforced Concrete—Typical Systems—Pile-driving—Sustaining Power—Various Data.

Structural Principles.—Maritime structures are, generally speaking, based on one or other of two distinct systems of construction. First, there is the compact, solid mass, capable of withstanding the attacks of the elements by means of sheer intrinsic inertia, and, secondly, there is the framework structure, composed of an association of members or parts, all slender in themselves, but so contrived and connected as to afford one another mutual support, and at the same time able to discharge special individual functions.

Typical of the former system is the breakwater, mole, or quay, built as a solid mound or mass of rubble, masonry, or concrete, or a combination of these, the ideal being a homogeneous monolith, without break or joint. This type is really an adaptation of nature's own system, exemplified in rugged cliff and massive headland.

The framework structure, on the other hand, is a strictly scientific design, utilising the minimum of material to the maximum advantage. It is based on the same theoretical considerations as those which govern the synthesis of all trusses, whether in the form of bridges, roofs, or other openwork. Its principal source of weakness lies in the jointing together of the various parts, for under the violent alternations of impact and recoil, which are characteristic of marine forces, there is every disposition for the joints to become loosened through excessive vibration. There is, moreover, another disadvantage attending those structures which are composed of unprotected metal and timber—viz., their liability to corrosion and decay. Both these considerations militate greatly against the realisation of any great degree of durability and permanence, and render structures of the second class inferior in certain respects to those founded on the former system, while, at the same time, they obviously involve much greater expenditure in the way of maintenance and repair.

There are, however, circumstances under which framework structures become inevitable, and many others where they are undoubtedly desirable. Thus a solid pier inevitably deflects the course of a littoral current, thereby diverting navigable channels into unknown directions, and bringing about

physical results which it is not possible to forecast with any certainty. A columnar pier, on the other hand, offers very trifling obstruction to current flow, and practically leaves the coastal régime unaltered. On these grounds, it has been deemed politic, at Zeebrugge for instance, to construct in open-work the portion of a projecting mole which immediately adjoins the shore, while that portion which lies beyond the range of the littoral current, or which is not likely to offer any injurious opposition to the motion of the sea, is built in the solid.

Framework, as adapted to maritime situations, consists of two distinct parts: the supporting columns or piles and the superstructural trussing. With the first of these we propose to deal in this chapter. On the second it will be necessary to touch but lightly, as the principles upon which it is based are common to all branches of engineering work, and in no sense can it be considered as a special feature of harbour engineering operations.

Piling is the term applied to all columnar members driven vertically, or nearly so, into the ground to form a foundation for constructional purposes. It includes two varieties: first, **sheeting piles,** which are employed to inclose or confine an area, and secondly, **bearing piles,** which act as isolated supports.

Sheeting piles are often much wider than they are thick, and are set with their edges in close contact, so as to form a continuous wall or partition. In order to achieve this result, they are driven in bays of moderate length, between leading or guide piles, to which horizontal walings are affixed. *Bearing piles* are more equilateral in cross-section, and are driven quite separately, or in clusters. Sheeting piles are provided with a knife edge at their lower extremities; bearing piles have either pointed or butt ends.

The materials from which piles are made are extremely varied, and include timber, iron and steel, concrete and reinforced concrete.

Timber Piles.—Up to within comparatively recent times, timber piles were most extensively, if not universally, in vogue, and they are still preferred in many cases where circumstances are favourable to their employment. They have been adapted to purposes both of a temporary and of a permanent nature. For the former class of work, they are still in general demand, but for the latter class they are now only utilised when considerations of economy outweigh all others. For jetties and piers destined to wear and rough usage, the durability of material, composed of metal and mineral which is practically indestructible, gives it an enormous advantage over perishable fibre; but for temporary work, such as gantry staging, cofferdams, and the like, the cheapness and adaptability of timber confer upon it certain advantages.

The character of the timber employed in harbour work depends upon the probable or estimated duration of its services. When utilised for permanent structures, only the best, hardest, and soundest timbers are admissible. In other cases, softer and less durable wood will suffice, provided it be kept under constant supervision and renewed whenever necessary.

So far as soundness and strength are concerned, there are few trees which are incapable of supplying logs and balks of a thoroughly satisfactory character. In harbour work, however, durability is the crucial consideration, and the conditions attaching to that qualification are much more exacting than those which govern the choice of suitable timbers for constructive purposes elsewhere. The alternations of exposure to the atmosphere and submersion in the sea, due to tidal fluctuations, constitute in themselves a most fertile source of decomposition, such as is not experienced in any other environment nor associated with any other branch of engineering. And, as if this were not sufficient, there is allied therewith a most pernicious and deadly subjection to the mechanical attacks of insectile [1] borers, which infest the waters of most ports.

In addition to the question of durability, however, there are the subsidiary, but no less essential, considerations of available scantling, cost, and facility of supply, each of which demands the careful attention of the engineer.

It is not proposed to enter into any lengthy dissertation of a botanical nature on the very great variety of trees which are available for engineering purposes; it will be sufficient to confine our attention to details of a practical kind in connection with those comparatively few species which have obtained wide and general recognition in connection with maritime work. These may be enumerated briefly in three groups:—

I. Greenheart, Mora, and the Eucalypti. These woods are extremely durable and highly repellent of marine organisms.

II. Teak and Oak. These are also very durable, but subject to insectile attack.

III. Beech, Elm, and Pine. These are moderately durable, and they succumb easily to marine organisms.

By far the most important group to the harbour engineer is the first. To this we must pay greatest attention, leaving the other groups, though they comprise timbers of more extensive use, to be but briefly noticed.

Greenheart (*Nectandra Rodiæi*) is an American product, the tree being a native of Guiana and the adjacent states of the South American continent, where it grows very abundantly in tracts lying within a hundred miles of the coast-line. It is a wood of extreme hardness and durability, with a very fine and compact, though uneven, grain. Its resistance to crushing is enormous, but it is very brittle and it splits under the least provocation. Before sawing, logs have to be bound very tightly with chains and wedges on each side of the projected cut; otherwise there is great danger of splitting, and a crack once started is prevented from extending with difficulty greater than that of the avoidance of the danger in the first instance.

[1] Objection may be taken to the use of this word in this connection. It is difficult, however, to find an accurate generic name for all varieties of these pests. The term insect is applied under license which is justifiable, since no confusion is likely to arise from its use.

Greenheart contains a poisonous oil, which renders necessary considerable circumspection on the part of carpenters and others engaged in dressing it. A splinter in the flesh almost invariably produces blood-poisoning, and the merest scratch should be promptly sucked and washed in clean water.

The weight of greenheart ranges from 60 to 75 lbs. per cubic foot, so that it has practically no flotation. This characteristic facilitates its manipulation for piling purposes, as it sinks readily into position. It can be obtained in balks from 12 to 24 inches square and up to 70 feet in length. It has an ultimate compressive strength, in short prisms, of 8 to $8\frac{1}{2}$ tons per square inch, and a beam of unit dimensions—*i.e.*, 1 inch square in section and 1 foot between supports—will fail at loads ranging from 950 to 1,500 lbs., centrally and concentratedly applied.

The colour of greenheart ranges from green to almost black.

Purpleheart is a wood of the same kind, from the same locality, with a difference only in colour, as indicated by the name. It is perhaps a little tougher and slightly more durable, but, on the other hand, it is not so readily procurable. Balks can be obtained up to 30 inches square.

Mora (*Mora excelsa*) is also a native of Guiana, but is a light-red wood, with several distinguishing characteristics. It shares the strength and durability of greenheart, while it differs from it in possessing great toughness and in lacking any disposition to split or splinter. It is rather lighter in weight, too, than greenheart, weighing from 57 to 68 lbs. per cubic foot.

The Eucalyptus family is a numerous one, and indigenous to the Australian continent.

Jarrah (*Eucalyptus marginata*) is a timber found in abundance in Western Australia, and, from its resemblance to mahogany, it is sometimes called Australian mahogany. It is hard, heavy, and close-grained; very liable to warp and split. It is also beset with clefts filled with resinous matter, which is sometimes found to be in a state of decay. The fibres contain an acid having a pungent odour. The tree grows to a height of 200 feet and more, but sound logs are limited to 40 or 45 feet in length and 12 to 24 inches square.

"Jarrah can vary very widely in durability and resistance. Conditions of growth affect its properties. The best timber is found among the rocky boulders and summits of the granite and ironstone ridges of its habitat. Trees grown in the sandy plains near the sea yield inferior timbers, twisted, shorter in grain, and less durable. The safest assurance of receiving sound Jarrah, when specified, is to have inspection made by the Government of Western Australia."[1] This is done at a charge of 3d. per 100 superficial feet. Jarrah is shipped from Freemantle, Rockingham, and Banbury.

[1] Armstrong on "Important Piling Timbers of Australasia," *Engineering*, 17th Nov. 1916.

The weight of jarrah is just about equal to that of an equal volume of water. It has little more than half the crushing strength of greenheart, and the ultimate transverse strength of a unit beam (1 inch square and 1 foot clear span) is between 500 and 650 lbs., concentrated at the centre.

Karri (*Eucalyptus diversicolor*) is a hard, heavy, straight-grained wood, with some claims to toughness. It is somewhat stronger than jarrah, and drives better, but is less durable in damp situations; though, when totally and continuously immersed, it is said to last well. In the log there is much resemblance between karri and jarrah. Karri, however, is deficient in the astringent gum which is characteristic of jarrah.

The **Blue Gum** (*Eucalyptus globulus*) and the **Stringy Bark** (*Eucalyptus obliqua*) are two varieties of the same species, which have latterly come into use and have demonstrated considerable merit for staging purposes in connection with the improvement works at Dover Harbour.

The former is so named from the characteristic glaucous blue tint of the young plant, though the colour of the mature wood is a golden yellow or brown. Both trees grow to an enormous height and girth, and furnish tough, strong wood, extremely durable under favourable circumstances, and more particularly in dry and open situations. Piles, 100 feet to 120 feet long and 20 inches square, have been obtained in Tasmania. Stringy bark, according to some authorities, weighs about 70 lbs. per cubic foot, and blue gum about 77 lbs.; others place the figures at 60 and 65 lbs. respectively. Some variation of weight in different specimens is, of course, inevitable. The transverse strength of unit beams, as above, may be taken at anything from 450 to 850 lbs.[1]

The **Turpentine Tree** (*Syncarpia laurifolia*), a native of North-Eastern Australia, also produces suitable timber for piles, which are driven with the bark on as a protection against attack by marine organisms.

" The accounts of the life of turpentine piles show considerable variation. At Sydney Harbour, after a long experience with turpentine, the same species was used for renewals. The new ones were 4,640 in number, ranging from 40 to 80 feet in length, and the engineer in charge was confident that they would give 30 to 35 years' service. However, the New South Wales authorities do not claim that turpentine is immune from the attacks of marine wood-borers, but they do assert that, with the bark intact, it is the only timber they dare drive unprotected by sheathing. Under the precautions observed it evidently gives satisfaction in those waters." [2]

The weight of the turpentine varies from 57 to 69 lbs. per cubic foot.

[1] Other serviceable varieties of the Eucalyptus family include Grey Ironbark (*E. crebra*), White Ironbark (*E. paniculata*), and Red Ironbark (*E. siderophloia*, *E. sideroxylon* and *E. leucoxylon*).

[2] Armstrong on "Important Piling Timbers of Australasia, *Engineering*, 17th Nov. 1916.

There are two varieties: the red and the black, of which the former is the sounder and more reliable.

It will be noted that all the timbers in this group have a very high specific gravity, and this property is found to be very useful in connection with driving piles in water of any depth. The lighter kinds of wood have necessarily to be weighted at the lower ends, in order to cause then to assume an upright position suitable for driving.

As regards durability in marine situations, it cannot be claimed that any of the foregoing timbers are absolutely immune from the attacks of living organisms. On the contrary, there is distinct evidence that boring has occurred in each kind of wood, though it is apparent that there is no great attraction in these timbers when others are present in the neighbourhood. Greenheart appears to be least susceptible, possibly on account of the poisonous oil which it contains. At certain ports it exhibits no sign of any depredation whatever, but this may be due to the absence of the inimical agencies. Altogether as a class, the timbers are the least vulnerable of any which can be applied to marine work, and in many instances they have demonstrated extremely high resisting powers.

The second group includes timbers which, though durable enough in themselves, are much more subject to insect attack.

Teak (*Tectona grandis*) is a native of India, Burmah, Siam, and Java. It is a firm, durable wood, fine and straight in grain, and easily worked, though possessing a tendency to splinter. It contains an aromatic oil of a resinous nature, which, on exposure, coagulates to such a degree of hardness as to spoil the cutting edges of tools. The tree often attains a height of over 100 feet and sometimes a girth of 10 feet. It is usually imported in logs from 25 to 40 feet long and from 10 to 20 inches square. The weight of teak varies from 41 to 52 lbs. per cubic foot, and the transverse strength of a unit beam lies between 600 and 700 lbs.

Oak (*Quercus*) is found on both the European and American continents, as also—less commonly—elsewhere. The best is grown in Great Britain. The wood is firm, with a fine, straight grain, comparatively free from knots, and it is readily cleavable. Logs vary from 10 to 40 feet long and from 10 to 24 inches square. The longer logs come from America. Oak is heavier than teak, weighing from 49 to 61 lbs. per cubic foot; but it is not quite so strong—about 50 to 100 lbs. less in ultimate transverse strength. Oak contains an acid which corrodes iron, and is, therefore, destructive to bolts and other fastenings.

Both the above timbers are admittedly assailable by insects, but they offer greater resistance and attain a higher degree of exemption than do members of the third and last group.

Elm (*Ulmus*) and **Beech** (*Fagus sylvatica*) are two well-known timbers, to which the term durable is only applicable provided the conditions be those of total immersion or continuous dryness. The weight of elm is about

35 lbs., and the weight of beech about 48 lbs. per cubic foot. As regards strength, beech has the superiority, being half as strong again as elm. The mean ultimate transverse load on a unit beam of elm is 400 lbs.; that of beech, 600 lbs.

Pine and **Fir** include a number of varieties of timber, some of which, such as pitchpine and Oregon pine, are highly serviceable to the harbour engineer for temporary staging and dams. Their durability under exposure to water is not very great, unless it be assisted by some treatment, as creosoting, which also affords protection to a certain extent against living organisms. These timbers must needs, however, be under constant supervision and inspection, and it is certainly not desirable to set them in positions difficult of access nor to place too great confidence in their capabilities of resistance.

Pitchpine (*Pinus rigida*) is obtained from the southern states of North America. It is a highly resinous wood, reddish or reddish-brown in colour. The resin in its pores renders it hard and difficult to work, but also increases its durability. The strength of pitchpine is often reduced by the practice of "bleeding" the growing tree—that is, tapping it for the turpentine which it contains. Logs are obtainable from 10 to 18 inches square and up to 60 or 70 feet long. The commonest sizes for piling purposes are from 12 to 15 inches square and from 40 to 50 feet long.

Oregon Pine or **Douglas Fir** (*Abies Douglasii*) comes from the North-West of North America. It has a light reddish colour, and is obtainable in logs up to 20 and 24 inches square and up to 100 feet in length. It is not so strong as pitchpine, but, affording larger sizes, is useful in certain situations.

In a communication dated 9th November, 1916, the Agent-General for British Columbia writes:—"The three great producers of Douglas fir are the Canadian Province of British Columbia and the American States of Washington and Oregon. The wood from the latter has never ranked so high in the timber trade as that from British Columbia and Washington, the timber from the more northerly areas being of a finer growth, and there has usually been a difference in price of from 6s. to 12s. per standard between the timber of Oregon and that of British Columbia and Washington."

Destruction of Timber.—The utility and value of timber being so greatly affected by its liability to destruction and decay, it is necessary to consider the sources of deterioration and the possibilities of their avoidance or cure.

Ravages by marine organisms claim first attention, as they constitute the most serious and pressing danger to which timber piles are exposed. Woods of the utmost durability in regard to chemical changes succumb only too rapidly from purely mechanical causes.

Of molluscs which attack timber immersed in a marine environment there are three classes of offenders, representing different families of two

species. These are *Teredo, Xylotrya,* and *Xylophaga,* somewhat in the order of importance.[1]

Teredo navalis.—This mollusc, one of the most pertinacious assailants of marine timber structures, is a member of the family *Teredinidae.*[2] It is an inhabitant of Atlantic and Mediterranean waters, and being transferred in ships' bottoms to the tropics and elsewhere, gives evidence of its presence in a considerable number of seaports. It has a decided preference for clear salt water, and deliberately avoids water which is muddy or sewage-polluted, or even fresh.[3] The process of its depredations appears to be as follows. Its eggs, drifting in the water, adhere to any exposed woodwork against which they happen to be washed by the sea, and there remain till ripe for hatching. On leaving its egg, the young teredo attacks the wood in its immediate vicinity by boring or tunnelling into it, principally in the direction of the grain. The holes, or galleries, increase in size with the growth of the animal, and they are lined throughout with a chalky secretion forming a thin, hard, smooth shell. It is no uncommon experience to find holes $\frac{1}{2}$ inch or $\frac{3}{4}$ inch in diameter, and the teredo has been known to attain a length of as much as 2 feet, though the average length is not more than 7 or 8 inches. Its operations seem to be chiefly confined within the tidal range—that is, between highest high-water mark and lowest low-water mark—but it also attacks timber at any moderate depth. At times it works with extreme rapidity. Some of the Memel fir piles of the old pierhead at Southend showed signs of the teredo within six months after completion, and in twelve months' time they were reported to be seriously injured. Fir and alder appear to furnish the most favoured fields for operation; oak and teak are less susceptible; greenheart and jarrah have a general reputation (not strictly maintainable) of being free from attack. Greenheart has been used for piles at the mouth of the Mersey without the slightest sign of deterioration of any part, even after the lapse of many years; but at Bombay the same wood has been freely ravaged.

Xylotrya is a genus equally destructive to timber piles, and its depredations are often mistaken for those of *Teredo,* from which, however, it is quite distinct. In many waters it is more numerous than *Teredo;* one very conspicuous species is *X. gouldii.* When this enters the wood, it is still a very minute bivalve embryo, but immediately afterwards it undergoes a remarkable change, the bivalve shell becoming modified to form boring apparatus with a surface covered with rows of tooth-like protuberances similar to a rasp. The *Xylotrya,* after excavating the fibre, swallows it through a wide

[1] Mr. Howard Weiss in his "Preservation of Structural Timber" (1915) also includes *Nausitoria* as being very active on the North Pacific Coast (p. 23).

[2] At one time it was classed with the *Pholadidæ,* but, latterly, it has been raised to the dignity of a family of its own.

[3] Mr. Robson, of the British Museum, informs me that a *Teredo navalis* occasionally finds its way upstream to fresher water.

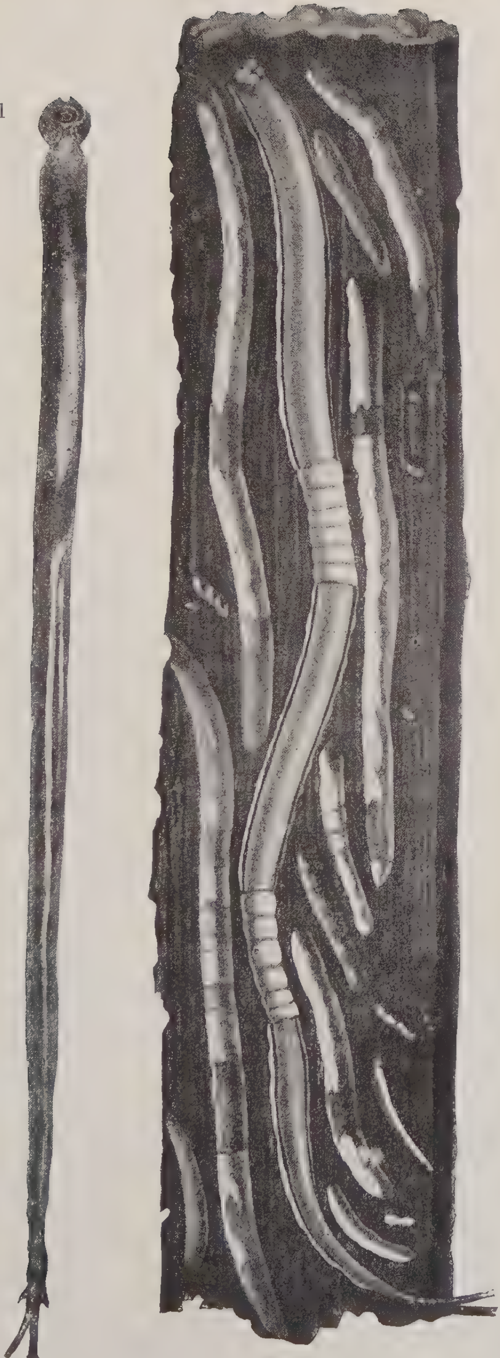


Fig. 76.—Views of *Teredo navalis*, detached and embedded in Timber. *From Mémoire sur la Conservation des Bois à la Mer*, by August Forestier, Paris, 1868.

pharynx and ejects the macerated material by means of a siphon-like appendage at the rear. The siphons, two in number, project out of the borehole and circulate a stream of water within the body of the borer. The *Xylotrya* is very active on the Atlantic Coast of North America.

The *Pholas dactylus* is a member of the *Pholadidæ* family. On the whole, it evinces a more pronounced taste for mineral substances, such as limestone and sandstone; but it also turns its attention to woodwork, which it honeycombs by boring a number of holes very closely together. Compared with *Teredo*, however, it is of small importance, so far as timber, at any rate, is concerned. The animal attains a length of 4 or 5 inches.

The third boring tribe of similar habits and tendencies is the *Xylophaga*. Its members are small in size, and they do not line their excavations.[1]

The *Chelura terebrans* (*Amphipoda*) (fig. 77) is a small crustacean somewhat resembling a minute shrimp, both in shape and colour. It is very small, not

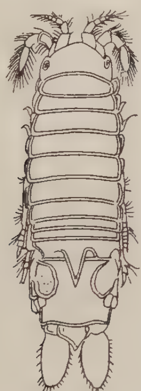


Fig. 77.—*Chelura terebrans* [2] (Magnified).

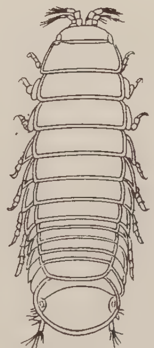


Fig. 78.—*Limnoria lignorum* (Ratke)[2] (Magnified).

more than $\frac{1}{4}$ inch in length. In addition to feet, it is provided with a pair of limbs, near the tail, which it employs in leaping. The *Chelura* destroys wood by cutting or tearing it away in thin flakes, working inwards from the exposed surface. It manifests a decided partiality for pure, clear seawater, and is consequently more often found along the open coast than in inclosed basins and harbours.

The *Limnoria lignorum* (*Isopoda*) is another lilliputian, whose length seldom reaches to more than $\frac{1}{6}$ inch. In size and general appearance it is not unlike a grain of rice. It is mainly troublesome on account of the vast numbers in which it infests certain localities, and, as it is indifferent to the foulness or otherwise of the water, no harbour precincts can be considered free from its presence. The *Limnoriæ* are active mainly about

[1] The evidence on this point is somewhat contradictory.

[2] From "An Account of the Crustacea of Norway," by G. O. Sars, Professor, University of Christiania (1895).

and below high water of neap tides, depredations proceeding rapidly until the whole of the timber-work is eaten away. Large balks of unprotected fir have been completely destroyed in three years, and even creosoted timber has perished within a decade.[1]

The attacks of the white ant (*Termes*) in tropical countries do not call for detailed mention. They are not particularly associated with maritime situations, and submerged timbers are, of course, not affected in any way.

Decay of Timber.—Apart from the mechanical destruction of timber, there is the question of natural decay, which is due to one or other of two distinct forms of decomposition, known respectively as *dry* and *wet rot*. The former, which is a process of fibrous disintegration, accompanied by the growth of a parasitic fungus, is attributable to, and certainly accelerated by, the absence of adequate ventilation. The woodwork attacked is mostly that which is situated in confined and stuffy places, to which air has insufficient access—conditions not generally allied with harbour work.

Wet rot, on the other hand, has a much more general and appropriate connection. It is the most characteristic disease, in fact, to which marine timbers are liable. It arises from, and is promoted by, frequent alternations of dryness and moisture, and these conditions are obviously prevalent along the water's edge. Every time a log becomes immersed and dries again, a fresh portion of the fibre is converted into soluble matter, which, in due course, is abstracted and lost. Furthermore, the continual evaporation of moisture from the pores of the wood results in putrefaction, the progress of which, once commenced, is often rapid.

Wet rot will attack indifferently any part or substance of a log, whether it be heartwood or sapwood; whereas dry rot is generally to be found in the latter only. The disease, moreover, is contagious, and affects adjacent timbers which may not in themselves be exposed to the same predisposing causes.

Preservation of Timber.—Having enumerated and described the inimical agencies, we come now to the means used to combat them. Expedients, as diverse as they are multitudinous, have been tried from time to time with a view to increasing the durability of timber, both as regards preserving it from internal decay and protecting it from external attack.

The commonest preservative for structural work is **paint.** Applied to seasoned timber completely deprived of free sap, the method is one of the most efficacious which can be devised; but it calls for frequent and regular renewal, and this, in the case of submerged work, is an insurmountable obstacle to its adoption. Moreover, sea-water tends to soften paint, and the

[1] The destruction of the Oregon piles supporting the temporary staging for the reconstruction of the Tyne North Pier was particularly noticeable, as much as 4 inches having been removed from each side of a pile in nine years, a rate of nearly ½ inch per annum. Barling on Tyne North Pier, *Min. Proc. Inst. C.E.*, vol. clxxx.

chafing of floating objects against the surface of the wood soon wears away its protective coating. The same objections apply to other substances, such as *tar*, *verdigris*, and *paraffin*, which have either been used or proposed as a substitute for paint.

Creosoting.—The best and only really effective agency for increasing the longevity of timber work in contact with moisture is the process known as **creosoting.** It also acts as a deterrent to marine borers, though not perhaps to the extent of rendering the wood absolutely invulnerable. The principle is that of impregnating the wood with a preparation of coal tar so that the pores become filled with an antiseptic, bituminous substance, which excludes air and moisture, repels the lower forms of vegetable and animal life, and prevents putrefaction and decay.

Creosote is an oily liquid contained in the second distillation of tar, from which the ammonia has been expelled. Its composition is somewhat variable and very complex, so that a strict definition of its constituents is not feasible.[1] Most of the present-day specifications require that the creosote should remain liquid at 90° F., and that it should not deposit more than about 20 per cent. of naphthalene at 60° F.[2]

It is essential to the efficacy of the treatment that, as a preliminary, as much of the moisture as possible should be abstracted from the interior of the timber. This is usually effected by air-seasoning, the material being stacked in the open air for periods ranging from 3 to 20 months, according to the class of timber. After thorough seasoning, the logs to be treated are run into cylinders, from 100 to 150 feet in length and 6 or 7 feet in diameter. The cylinders are closed and filled under pressure with creosote heated to a temperature of 170° to 190° F. In some cases a vacuum is created in the cylinders, prior to the admission of the creosote, for the purpose of extracting the air from the pores of the wood.

The alternative method of seasoning by steaming, though a much more

[1] "Creosote oil, in the scientific sense, may be properly defined as any and all distillate oils boiling between 200° and 400° C., which are obtained by distillation from tars consisting principally of compounds belonging to the aromatic series, and containing well-defined amounts of phenoloids."—Report by Committee on Preservatives, Eleventh Annual Meeting of American Wood Preservers' Association, 1915.

[2] The following is a standard specification adopted by the American Railway Engineering and Maintenance of Way Association :—"The oil shall be the best obtainable grade of coal-tar creosote—that is, it must be a pure product of coal-tar distillation, and must be free from admixture of oils, other tars, or substances foreign to pure coal tar ; it must be completely liquid at 38° C., and must be free from suspended matter. The specific gravity of the oil at 38° C. must be at least 1·03. When distilled according to the common method—that is, using an 8-oz. retort, asbestos-covered, with standard thermometers, bulb 1½ inches above the surface of the oil—the creosote, calculated on the basis of dry oil, shall give no distillate below 200° C., and not more than 5 per cent. below 210° C., nor more than 25 per cent. below 235° C., and the residue above 355° C., if it exceeds 5 per cent. in quantity, must be soft. The oil shall not contain more than 3 per cent. of water."

rapid process, is attended by grave risk of injury to certain classes of timber, producing shakes and other defects, and, in the case of Oregon pine, it is alleged, reducing the strength by one-third.[1] The logs are placed in the cylinder in their unseasoned condition, and there subjected to heat by the admission of steam, at such a rate as to secure 20 lbs. pressure within 30 to 50 minutes. The pressure is maintained for periods of one to five hours, depending upon the character of the timber. When the timber and its contained moisture have fully acquired the temperature of the steam, the latter is withdrawn and a vacuum created. This produces rapid and intense evaporation, and causes the wood to part with a very considerable portion of the moisture. At the end of half an hour creosote is admitted to the cylinder without otherwise disturbing the vacuum, after which pressure is applied as before, and, in both cases, remains in operation until the desired degree of absorption is obtained, or the wood is unable to take any more fluid.[2]

A third method is that of immersing the green timber in a cylinder charged with hot creosote, which is kept at a temperature above the boiling point of water, generally from 215° to 225° F. Boiling is maintained for four to eight hours, and the water vapour is condensed by a surface condenser. If, as in Sir S. Boulton's process, a vacuum is created, it is not necessary to maintain so high a temperature, and there is correspondingly less risk of damage to the timber. The treatment is employed in America for Oregon pine.

Owing to the high price of creosote an endeavour has been made to economise in the quantity used by removing as much as possible from the wood after impregnation. This is known as the Rueping[3] process, but it is not to be recommended for marine work, as it is desirable to charge timber piles with the maximum quantity they are able to retain.

Burnettising is the term applied to a process of treatment with a solution of chloride of zinc, containing $2\frac{1}{2}$ to 3 per cent. by weight of the chloride. Ordinary immersion will suffice, but impregnation is expedited by using pressure. The amount of absorption ranges from ·3 to 2·7 lbs. of dry salt

[1] Report by H. B. MacFarland to American Railway Engineering Association, August, 1914, quoted in *Proceedings of American Wood Preservers' Association*, 1915.

[2] Mr. Fergusson, General Manager of Messrs. Burt, Boulton & Haywood, Ltd., has kindly supplied me with the following data as to limits of absorption :—Baltic fir or Scotch fir (*Pinus silvestris*), with all the sapwood on, will take as much as 20 lbs. of creosote per cubic foot ; ordinary fir (*Abies*) and spruce (*Picea*) after long seasoning, 8 lbs. ; Oregon pine (*Abies Douglasii*) with all the sapwood on, and after special treatment, about 10 lbs. ; oak generally about 5 lbs., but some qualities not more than 3 lbs. ; beech from 15 to 20 lbs. The Eucalypti about 2 lbs.

[3] Sometimes called the Empty Cell Process, in contradistinction to the Ordinary, or Full Cell, treatment. The Lowry process is also based on the same principle. The Rueping treatment requires an initial air pressure to react in expelling the surplus creosote. With the Lowry process, a quick, high, final vacuum is depended upon to recover the oil.

in various timbers.[1] The treatment is only suitable for deck structures or for those parts of jetties and wharfs which are not in direct contact with stagnant or flowing water, as the salt is liable to be dissolved and carried away.

Boucherie's Process consists in impregnating timber with a solution of sulphate of copper (1 per cent. by weight) in water. The usual course of procedure is to cap one end of a log in a water-tight manner, and then allow the liquid to penetrate the pores to the other end, so displacing the sap, under a head of 30 to 40 feet, which produces a pressure of 15 to 20 lbs. per square inch. The extent to which penetration takes place can be tested by the application of prussiate of potash: whenever this substance comes into contact with sulphate of copper a brown stain is left.

Timber is **kyanised** by immersing it in a solution of corrosive sublimate (perchloride of mercury) contained in a wooden tank. The strength of the solution varies from $\frac{1}{3}$ to 1 per cent. by weight, according to the porosity of the timber.

None of the last three methods has proved so effective, or come into such general use, as creosoting. In fact, it is doubtful whether any of them is of the least benefit in warding off insectile attack, and this, in maritime situations, is an object no less important than the preservation of timber from decay.[2]

The only apparently completely successful way in which timber may be

[1] A Committee of the American Wood Preservers' Association recommended that "a full impregnation which will insure the retention of a minimum of one-half ($\frac{1}{2}$) pound of dry salt per cubic foot of timber be given. In stating the above minimum, standard practice is followed, but the Committee suggests that under usual conditions a three-quarter ($\frac{3}{4}$) pound injection would be advisable."

[2] The following extract from a recent issue of *Engineering* should, however, be noted:—

A process invented by Mr. J. E. Cunningham, of Sydney, and known as the *Carbo-Teredo Process*, consists in impregnating timber with a hydro-carbon and in producing a close and even charred surface on the outside by means of a high-pressure gas flame. The charred stratum is not superimposed. If any portion of this stratum gets damaged by handling in driving, the damage is made good before the timber is submerged. Knots and openings, as well as the ends of timbers, are submitted to re-impregnation and re-charring, whilst joints and checkings are but slightly charred on both surfaces so as to admit of tight fitting.

Experiments have been made with piles of Oregon pine and ironbank in Sydney Harbour spread over a period of two years (1914-16), and it is reported that the untreated timbers were destroyed by organisms while the treated timbers remained intact.

The treatment of the piles by the carbo-teredo method cost about 11s. per pile.—*Vide* "Preservation of Timber from Boring Organisms," *Engineering*, Feb. 23, 1917.

There is also a form of treatment involving the use of "Carbolineum Avenarius," referred to in a paper on Galvan Port, Bahia Blanca, Argentine, read before the Institution of Civil Engineers in November, 1915, by Mr. C. A. Trery, of which the author has been unable, owing to the death of Mr. Trery, to obtain full particulars.—Vide *Min. Proc. Inst. C.E.*, vol. cci.

guarded in this respect is by means of some external covering excluding the wood from actual exposure.

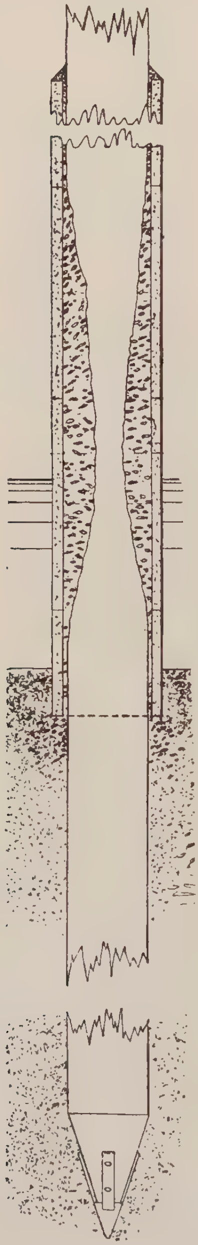


Fig. 79.—Application of Reinforced Concrete Slabs to Decayed Timber Pile.

Sheathing is a protective device which consists of enveloping a pile in a covering of metal, earthenware, or other material impenetrable by insects. A thin covering of copper plates has proved satisfactory in repelling worms from piles, when the covering has extended from below the mud level to above high water mark; otherwise the insects intrude themselves between the metal and the wood. The method is obviously an expensive one, and therefore, not likely to commend itself for general adoption. Zinc has been tried as a substitute for copper, but it is soon corroded by sea water. Muntz metal is another substitute, but its application has been too limited for definite judgment of its powers. Studding with broad-headed scupper nails is an old expedient, the principal drawback of which is its troublesomeness, and, of course, its expense.

Earthenware pipes, such as ordinary drain-pipes, and cylindrical casings of wire netting bedded in concrete, are efficient preservatives of piles in situations free from shocks, collisions, and erosion. The space between the pipe and the pile must be filled with sand or cement grout. A simple coating of Portland cement has been tried, but the film is too thin and easily cracked. Lately, a system of facing wooden piles with reinforced concrete slabs has been promoted by Mr. Cooper Poole, the harbour engineer of Southampton. The slabs are primarily intended for application to piles which are in such a state of dilapidation as to call for renewal or repair. At each corner of the pile a small angle iron is spiked on to the timber so as to form a guide for the slabs. These last are connected up and allowed to sink into the mud until they take a bearing, when the inclosed space is filled with concrete. The system, however, is applicable also to piles which are whole and perfect, as a preservative. On the Pacific coast a wrapping of jute burlap, in combina-

tion with a preparation of paraffin, powdered limestone, and kaolin, is reported to have achieved successful results.

From the foregoing details, it is obvious that the use of timber piles, though convenient, is attended by a number of serious disadvantages. There can never be any complete sense of security in reference to the part they play in permanent structures, and the increasing scarcity of logs of a suitable size, together with the difficulty of obtaining them at a moderate cost, has led to the introduction of piles composed of metal entirely or of metal and concrete combined.

Metal Piles.—Metal piles are ordinarily either of wrought iron or steel. The pointed or driving end is frequently cast, but, generally speaking, cast iron is of too brittle a nature for use in the shank of a pile, unless special precautions be taken in driving, or the ordinary method of impulsion by a falling weight be replaced by some other system. Thus, with screw ends, cast iron tubes or pipes are often used instead of timber logs (which are equally available), the means of forcing into the ground being rotation round the vertical axis. This constitutes, however, a method of treatment so distinct and exceptional that it may be regarded as not affecting the general question.

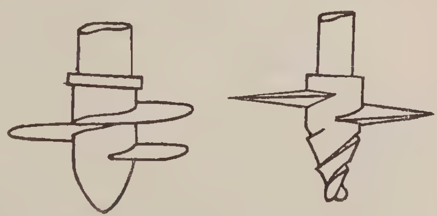


Fig. 80.—Screw Pile Bases.

For the sake of dismissing it from further consideration, it is convenient to introduce here a few explanatory words concerning the system of screw piles. The screw end consists of a broad blade, forming, in most cases, little more than a single turn or a turn and a quarter. It has the property, therefore, of furnishing a base of much greater area than that afforded by the ordinary pile, and on this account is useful for foundation work in compressible strata, where it is desirable to spread the load over as large an area as possible. Moreover, there is an absence of vibration in the process of driving, which is a distinct advantage. The piles are driven by means of a capstan head or a drum of large diameter temporarily bolted on to the shank, and raised from time to time as the rate of driving requires. In the former case, capstan or lever bars are used; in the latter, a winch, to which is led a wire rope wound round the drum, supplies the motive power. In primitive and isolated cases, animal labour has been utilised.

Steel or **wrought-iron piles** partake of all the recognised forms emanating from manufacturers' rolling-mills. Channel and joist sections are most common. Such piles, though available for solitary positions, are more generally found in close association, as sheet piling. When this is the case, a certain, and by no means negligible, amount of mutual interdependence and support is afforded by binding intimately together the adjacent edges of the

piles. This can be done by forming a series of grooves, preferably without the aid of rivetted connections, as exemplified in figs. 81 to 84, which represent typical sections manufactured by the British Steel Piling Company.

The interlocking arrangement is extremely useful in forming a water-tight inclosure for dams. Hydraulic pressure against the outer face will generally prevent the passage of water, but where any leakage manifests itself, it can easily be checked by sprinkling ashes, sawdust, or any light material of a similar kind, upon the outer surface, whence it will be sucked into the defective joint.

The driving of these piles is effected in the ordinary way by means of a

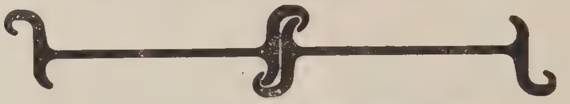


Fig. 81.—"Simplex" Steel Sheet Piling.

Fig. 82.—"Simplex" Steel Sheet Piling.

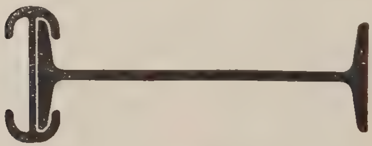


Fig. 83.—"Universal" Joist Piling.

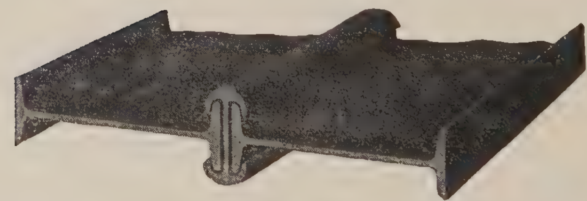


Fig. 84.—"Universal" Joist Piling.

falling ram; only, it is necessary to interpose a wooden "dolly"—a 6- or 8-foot length of greenheart timber—between the cap of the pile and the underside of the ram. The cap of the pile is a removable block or plate of cast steel, several inches thick, temporarily secured in position with the aid of bolts and removed after the operation of driving is finished.

Metal piles, though indestructible by insects, are subject to corrosion, with results equally disastrous in the long-run. The effects of oxidisation are most to be dreaded in the case of the outstanding piles of piers and jetties. All

ironwork immersed in salt water, and especially when alternately wet and dry, undergoes chemical changes subversive of its strength and durability. Hence the manifest necessity of providing it with some protection akin to that which is accorded to timber piling.

Of the methods in vogue for the prevention of corrosion in iron or steel, two stand out in greatest prominence—painting and galvanising. The former of these is only of the nature of a temporary preservative, and has to be re-applied at regular and frequent intervals; the latter cannot be renewed in the case *in situ* structures, and, though the initial treatment is understood to be more effective than painting, yet the environment of the seacoast is extremely detrimental to its durability.

The coatings applied to ironwork under the head of paint comprise those which are composed of red lead and those which have oxide of iron for their base. The latter of these has been advocated on the ground that it removes the tendency to galvanic action produced by two diverse metal substances in contact with one another in the presence of moisture. Other coatings are mineral or vegetable tar, black varnish, siderosthen, and various bituminous solutions. It is obvious that only the surfaces of piles which lie above the water level can be treated with those applications after erection.

For cast-iron work, and especially for cast-iron pipes, no better preservative could be devised than the *Angus-Smith treatment*, which consists in dipping the pipes while hot into a liquid mixture of coal-tar, pitch, linseed oil, and resin.

Iron and steel are *galvanised* by dipping them into a bath of molten zinc so that a veneer of the latter metal covers them completely. To effect this treatment properly, the surface of the metal treated must be absolutely clean and free from scale and grease. The process is effective against ordinary atmospheric influences, provided the zinc covering be maintained intact. If a crack or perforation occurs, corrosion sets in and proceeds rapidly. Against sea air and water, galvanising does not afford much protection.

It is obviously no simple matter, therefore, to find a satisfactory and reliable method for insuring the permanence of iron and steel work in maritime situations, and particularly in the case of piling, where the work is so difficult of access. The desired result, however, has been achieved by the ingenious expedient of enveloping the metal in concrete, and this brings us to the system of combined steel and concrete which now generally goes by the name of reinforced concrete.

Reinforced concrete consists essentially of a core or internal network of metal, completely embedded in concrete, so that no part of the metal is exposed to, or in contact with, any external atmospheric or aqueous influences. As applied to piling, the system has many and important advantages. Reinforced concrete piles are not subject to oxidisation, decomposition, or decay. Experience has demonstrated that steel bedded in Portland cement concrete does not rust even when immersed in water, and

that a rusty bar so treated manifests no increase in corrosion. Moreover, reinforced concrete piles do not offer the least incentive or attraction to sea organisms or insects; they are fireproof as well as waterproof; their durability is beyond question; they cost less than long greenheart piles, and little, if anything, more than creosoted pitchpine; they can be jointed, and lengthened or shortened at will; and, finally, their compressive strength and supporting power is very great.

Reinforced concrete piles vary considerably in design, according to the individual ideas of numerous inventors. It will only be necessary, however, to refer to a few of the better known examples, which are distinctly applicable to harbour work. The circumstances of foundation piles for

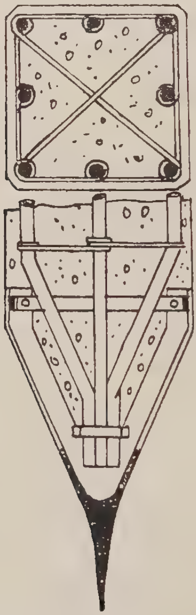


Fig. 85.—Hennebique Pile.

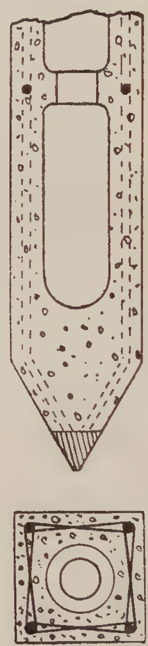


Fig. 86.—Mouchel Hollow Pile.

inland structures and for piers and jetties are by no means identical. It cannot fail to be evident that a pile driven wholly into the ground, as in the former case, need not possess the same lateral stiffness which must essentially appertain to a pile only partially buried, and subject, moreover, to the incidence of forcible impact throughout a very considerable part of its length.

Thus, for landwork, concrete piles may be formed by simply drilling or boring a hole within an iron shell or tube, and filling the latter with concrete, the shell in many cases being withdrawn as the work proceeds. This method, of course, is quite inapplicable to piling in water.

The **Hennebique bearing pile** (fig. 85) contains a series of long, round

bars, generally from four to eight in number, set parallel to, and arranged symmetrically around, the longitudinal axis of the pile. These bars are connected together and maintained in position by bonds, or ties, of iron wire and distance pieces. The bars vary from 1 inch to $1\frac{5}{8}$ inches in diameter, and the wire is usually $\frac{3}{16}$ inch thick. The distance pieces, which are about $\frac{1}{2}$ inch in diameter, with forked ends, are set at a normal distance of 10 inches apart, but at and near the top of the pile the distance is reduced to 2 inches. The toe of the pile is a pyramidal block of cast iron, into which wrought iron straps have been inserted. The upper ends of the straps are bent inwards towards the centre of the pile. The longitudinal rods of the pile are continued as far as the casting, being deflected to the required splay.

The **Hennebique sheeting pile** is made on the same lines as the bearing pile. There are three rows of longitudinal rods, arranged in pairs, and connected, as before, at 10-inch intervals with iron bands or clips. The ends of sheet piles are wedge-shaped, with a downward splay towards one side. This is extremely useful, during the driving process, in keeping a pile in continuous contact with its neighbour, towards which the resultant pressure on the splayed edge causes it to be urged. To complete the connection, piles are moulded with cylindrical grooves in each of their sides, one of which possesses a short spur or projection, capable of engaging in the groove of an adjoining pile. When two consecutive piles have been driven, their combined grooves form a cylinder which, after being cleansed by forcing water through it under pressure, is grouted with cement.

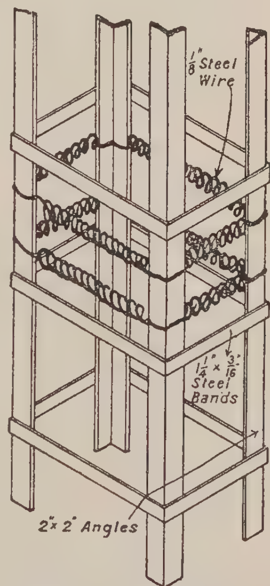


Fig. 87.—Johnston Pile.

A slight variant on the Hennebique pile is the **Mouchel hollow pile** (fig. 86). It has the longitudinal rods, wire ties, and distance pieces of the former, but, with the object of saving material and reducing weight, it is concreted with a core, which, being withdrawn, leaves the pile hollow. Diaphragms at intervals strengthen the concrete work. The Mouchel pile is light and easy to handle. The reduction in strength is such as to be practically inappreciable, and does not affect the utility of the pile.

The **Johnston pile** (fig. 87) differs from the preceding in that the longitudinal rods are replaced by angle bars at the corners of the pile. These are bound together by flat bands and coiled steel wire.

The **Considère pile** (fig. 88) consists essentially of a spiral reinforcement of rods enclosing the longitudinal rods. It is specially claimed for this system that the resilience is such that the pile does not require the interposition of

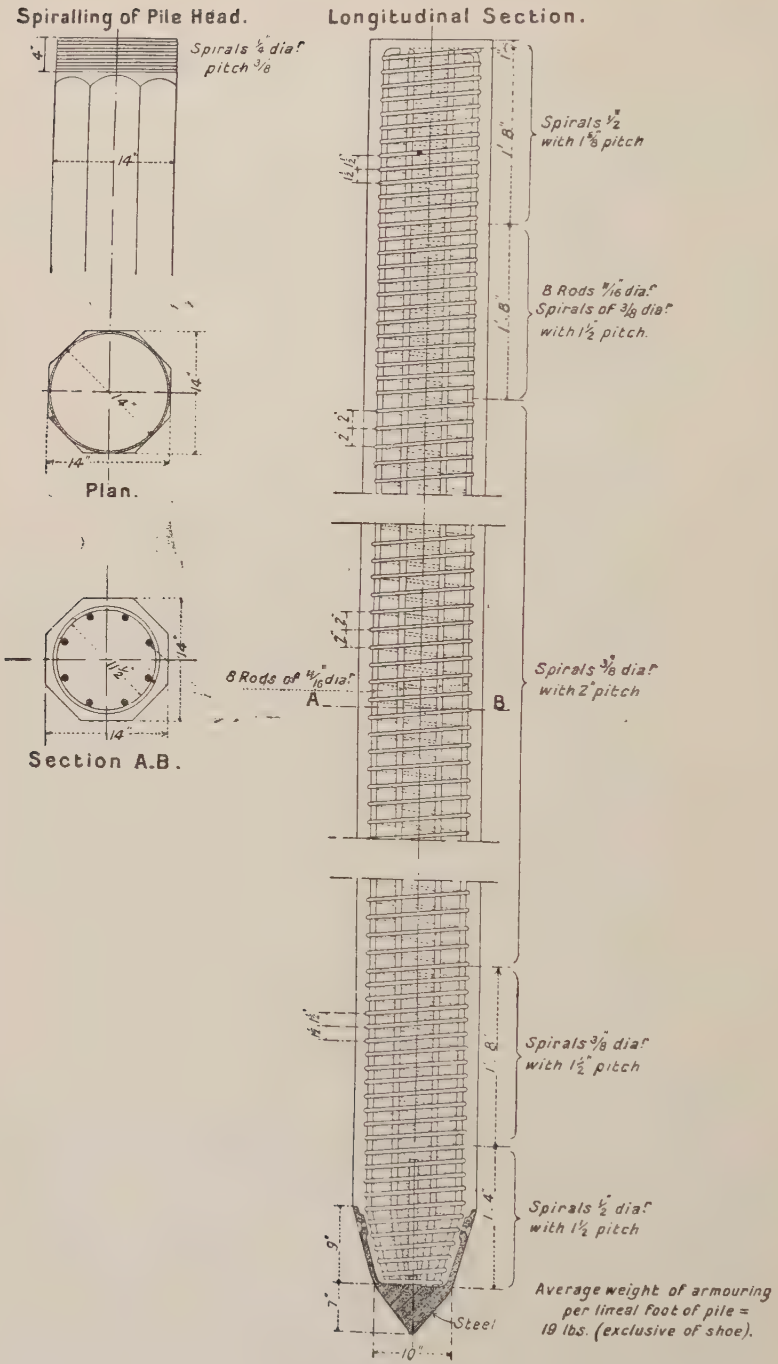


Fig. 88.—Considère Pile.

the usual "dolly," but is capable of receiving the direct blow of the pile-driving hammer without injury. Considère piles, 57 feet long, are stated to have been driven several feet into limestone rock by rams of 2 tons' weight falling through 6½ feet on to the unprotected heads of the piles.

The **Chenoweth pile** (fig. 89) is constructed on a similar principle. A

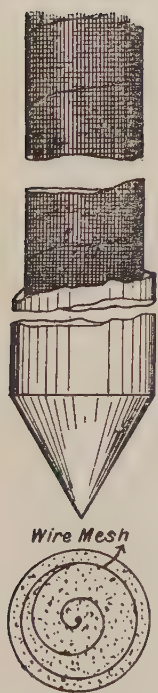


Fig. 89.—Chenoweth Pile.

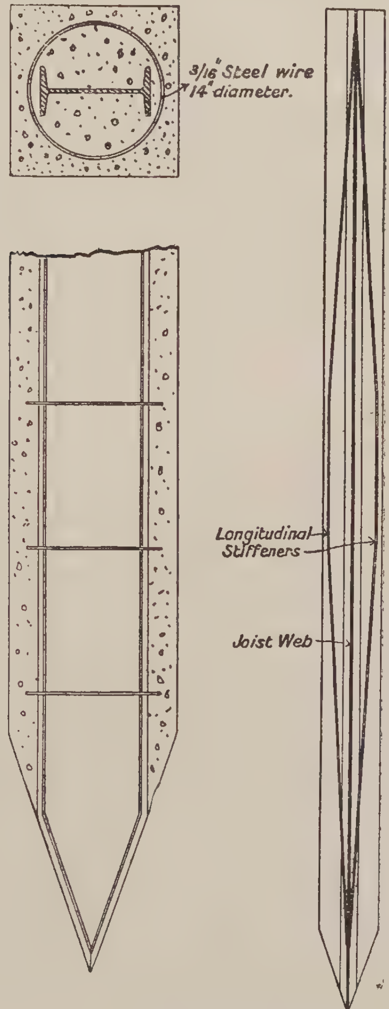


Fig. 90.—Williams Pile.

sheet of iron mesh is bent round a longitudinal axis in the form of a continuous spiral, forming a cylinder which is surrounded and filled with concrete.

The **Williams pile** (fig. 90) consists of a central rolled steel joist surrounded at intervals by steel wire hoops and having cambered longitudinal stiffeners.

Wooden Cylinder Piers.—An interesting combination of timber piling with timber and steel and concrete reinforcement has been practised for a number of years at wharfs constructed in San Francisco harbour and along the Pacific coast.[1] The piers are termed "wooden-cylinder" piers, but, as a matter of fact, the wooden cylinder does little more than perform the part of a temporary mould for the concrete, and it decays or is destroyed within a relatively short time.

The process consists in driving timber piles, either singly or in clusters, three being the usual number to each core or nucleus. These piles are driven so as to penetrate, to a depth of 50 feet or more, the mud forming the harbour bottom. Over each cluster, or core, is lowered a wooden casing, or cylinder, usually with an internal diameter of about 4 feet. The cylinder is made of staves of Oregon pine, 3 to 4 inches thick, bound together by iron hoops, provided with adjustable lugs. The hoops are spaced about 24 inches apart, and are so fitted as to render the cylinder watertight. After being driven into the mud to the extent of about 15 feet, the water is pumped out of the cylinder, together with some of the mud, leaving a difference of 2 to 5 feet in level between the mud inside the pipe and that outside. Next, in the annular space between the pile core and the timber cylinder is placed the metallic reinforcement, consisting of expanded metal or some similar metal web, thus forming a second cylinder about 12 inches less in diameter than the outer cylinder. The intervening vacuities are then filled in with concrete, and the operation is complete. The wooden covering lasts for at least four years, giving ample protection to the concrete until it is thoroughly indurated.

In a later type of this system of construction the wooden cylinder is replaced by a casing of reinforced concrete, the interior core being retained, but consisting only of a single pile. The interstitial web of metal is also omitted. The casings are 2 inches thick and 2 feet external diameter, and the concrete is of Portland cement in the proportion of 3 to 1.

Moulding.—Reinforced concrete piles may be moulded either vertically or horizontally. For the former method, it is claimed that it results in greater uniformity in density throughout any horizontal layer, while the latter method is characterised by greater convenience. The advantage gained by vertical moulding is of questionable validity—there is no reason why horizontally moulded piles should not be absolutely homogeneous—and, in any case, it cannot be said to compensate for the greater trouble of moulding in that way and the higher cost involved.

In *horizontal moulding* a box is formed of the dimensions of the pile, but without a top—that side being reckoned the top which comes uppermost when the box is laid flat on the ground. The sides of the mould are well soaped or oiled to prevent adhesion; then a layer of concrete is deposited in

[1] Report of the Engineers of the Federated Harbour Improvement Associations on San Francisco Harbour, 1908, and Crafts on Concrete Pier Construction on the Pacific Coast, in *Cassier's Magazine*, May, 1908.

the bottom to the extent of the outermost covering of the metal: that is to say, 1 or 2 inches, as the case may be. This is very carefully rammed and consolidated before the metal framework is laid upon it. The latter operation requires great care. The framework must be set perfectly true to the axis of the pile, and the shoe, with its bevelled sides, must be accurately adjusted and brought into close contact with the ends of the frame. The box is then carefully filled with concrete in a series of thin layers, deposited without a break, each layer being well punned and the concrete pressed into all corners, angles, and recesses. The top, or fourth side of the pile, is formed by striking the edges of the box with a straight-edge, so that the concrete just comes flush with them. The pile is left for a week in the mould, then the mould is removed and the pile allowed to harden, either in water or while constantly wetted. A month or six weeks elapses before the pile is ready for driving.

To facilitate lifting, a bolt hole is cast near the top of the pile. The bolt and a shackle enable the pile to be swung easily into position. The green pile, however, is not handled in this way, but by means of chain slings passing round the pile, the sides of which are protected by deals at the points of contact.

For *vertical moulding*, the box is set upright and the metal framework first placed in position with the shoe downwards. Concrete is then filled in to the mould and around the metal, as carefully as in the previous case. The pile is built up in series of layers from 4 to 6 inches in depth, the fourth side of each layer being formed by a batten fixed across the open face by fitting into grooves or being otherwise secured to the box, the whole height being treated in this way. The remaining operations are as before.

The materials used for reinforced concrete piles must be the best of their respective kinds. The concrete particularly calls for special attention. The proportions used lie between one part of Portland cement to four or five parts of aggregate, the latter compounded of gravel and sand in the ratio of 2 : 1. In one system (the Williams') the aggregate consists of clean shingle, which will pass through a $\frac{3}{4}$-inch gauge but not through a $\frac{1}{4}$-inch gauge, mixed with half its volume of sand. In Hennebique work the gravel is also sifted through two sieves. The first has apertures 1 inch square; the other has four uncrossed meshes per linear inch. The residue from the first sieve is thrown against the second, and equal parts taken of that which passes through the second sieve and that which fails to do so. After the pile has been removed from the mould, it is well to give it a coat of pure cement wash. This closes the outermost pores and renders the pile more highly impervious. The non-porosity of a reinforced concrete pile is obviously essential to its durability. It is only by the complete exclusion of moisture from the embedded steelwork that the latter can be maintained in a serviceable condition.

Pile-driving.—Piles are forced into the ground, or driven, by means of piling machines, which are actuated by hand or steam power. The exceptional use of the screw pile has already been noticed (p. 113). The impelling force

is commonly a heavy weight or ram, which is allowed to fall within vertical guides from any desirable height. In the hand or ringing machine, the weight rarely exceeds one-third of a ton, and the fall, 4 feet. In other appliances the weight and fall range from 15 cwts. and 10 feet to 3 tons and 4 feet respectively. A heavy weight and a low fall are preferable to a light weight and a considerable fall, owing to the greater oscillation resulting from the latter arrangement and the consequent jar in the delivery of the blow, which thus tends to injure and split the pile. In concrete piles the absence of vibration is of primary importance. Indeed, such is the care which has to be exercised to prevent rupture, that the pile head is capped in a very elaborate manner. A cast-steel helmet completely envelops the head, its interior being filled with sawdust and sacking. Between the helmet and the ram of the pile-driver is also interposed a wooden dolly, so that a very considerable proportion of the momentum of the blow is absorbed before it reaches the pile.

A much more efficient implement, where conditions admit of its employment, is the steam hammer. Blows can be delivered with great rapidity and effect. Timber piles driven by an ordinary weight machine to the utmost capability of the ram have responded readily to the steam hammer and have been forced to a considerably increased depth. Steam hammers are of two types. In the first, the piston is maintained in constant contact with the pile head, while the blow is administered by means of a heavy cast-iron cylinder, moving up and down under steam pressure. An average cylinder will weigh a ton and its stroke will be 3 feet. In the second type the cylinder is affixed to the head of the pile and the hammer is attached to the piston. The disadvantage attaching to machines of the steam hammer type is the leakage of moisture from the cylinder, which softens the head of the timber pile, or dolly, under impact, and reduces it to a pulpy state. This necessitates cutting and dressing a fresh head, otherwise the power of producing penetration is much impaired.

In driving through sand and sandy gravel, very excellent assistance has been derived from the use of the water-jet. A pipe led down the side of the pile to be driven, transmits water under pressure to the ground in advance of the pile, and maintains the former in a state of fluidity until the required depth has been obtained. Immediately after the withdrawal of the pipe, the sand consolidates firmly round the pile and there is no further tendency to sinkage even under load. Piles treated in this manner rarely have pointed ends, as a butt end affords greater bearing area without appreciably increasing the difficulty of driving. Indeed, the perpendicularity of a butt-ended pile is more easily maintained.

The limit of driving varies so strikingly according to local requirements that no precise figure can be assigned to it. Obviously, a pile may support a light load with ease where a heavier one would cause sinkage. With a ram of 1 ton weight falling through 10 feet, the pile may justifiably be considered

adequately driven when eight or ten blows fail to produce a depression of more than $\frac{1}{4}$ of an inch. This will indicate the attainment of thoroughly firm ground, and any further attempts at driving will only tend to shatter the pile. Timber pile-ends become "broomed" or splintered under an excessive amount of impact. Apparently easy driving, after a check, may be due to this cause, and there is no means of ascertaining the fact except by withdrawing the pile.

Sustaining Power.—Piles, if completely embedded and driven to the limits stated above, may be loaded safely to the extent of half a ton per square inch of the area of the pile section. Those in soft, muddy ground, sinking freely, say 1 inch per blow, at the end of driving operations should not be loaded with more than a tenth of a ton per square inch, and even then some slight settlement may be anticipated.[1]

Illustrative of the first class, the following instances may be cited of timber piles driven to refusal in various situations at the port of Liverpool :—

Nature of Superstructure.	Sectional Dimensions of Pile.	Sectional Area of Pile.	Length of Pile.	Total Load on Pile.	Load per Sq. Inch of Pile Area.
		Sq. Ins.		Cwts.	Cwts.
Riverside Warehouse, .	12″ dia.	113	40′ to 50′	1,568	13·88
Quay Cargo Shed, . .	15″×15″	225	40′ to 45′	2,296	10·20
,, . .	12″×12″	144	40′ to 45′	1,170	8·12
Riverside Warehouse, .	14″×14″	196	40′ to 45′	2,177	11·11

At the port of New York, the conditions in many instances are such as to be typical of the second class. Along the North River, for example, where most of the transatlantic liners are berthed, a firm stratum cannot be reached by piles 80 feet long. Such piles, therefore, are dependent on the friction of mud against their sides to support both themselves and the load they carry. And although, under circumstances of this kind, great sustaining power could hardly be expected, it is recorded that loads equivalent to 40 tons per pile have been safely carried. This figure, indeed, is very much in excess of the limit previously specified. At one-tenth of a ton per square inch a circular pile, 18 inches diameter in the butt (such as is commonly used at New York), would only support 24 tons and a 22-inch pile not more than 36 tons.

Mr. J. A. Bensel, a former engineer-in-chief of the Department of Docks and Ferries at New York, carried out some experiments on the limiting capabilities of the piles employed there.[2] A platform was built near the foot of Seventeenth Street, North River, and the piles to be tested, all about 80 feet long, were driven in four groups within the area of the platform,

[1] This figure relates to piles of ordinary size, say 12 inches square in cross-section. As a matter of fact, the supporting power depends upon the surface exposed to friction. and, therefore, is governed by the sectional perimeter of the pile.

[2] Bensel on Dock Work in New York Harbour, *Proc. Int. Eng Cong. St. Louis*, 1904.

and arranged as follows:—Group I., plain, unlagged piles. Group II., piles lagged with four pieces of 5-inch by 6-inch lumber, 30 feet long. Group III.,

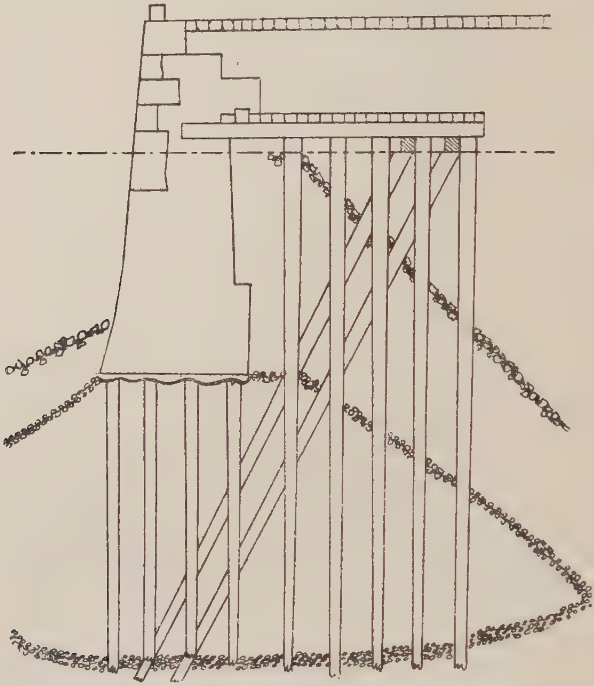


Fig. 91.—Piled Foundation to New York Quay.

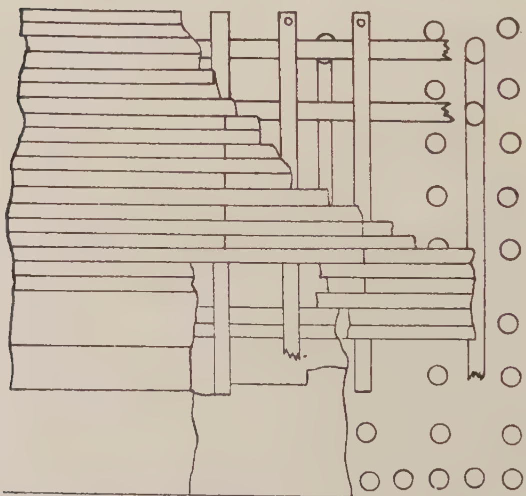


Fig. 92.—Plan of Piled Platform of Quay Wall, New York Harbour.

piles lagged as in Group II., but arranged in pairs, so as to obtain the effect of greater proximity, the piles in each pair being spaced 2 feet 8 inches apart.

To face p. 125.]

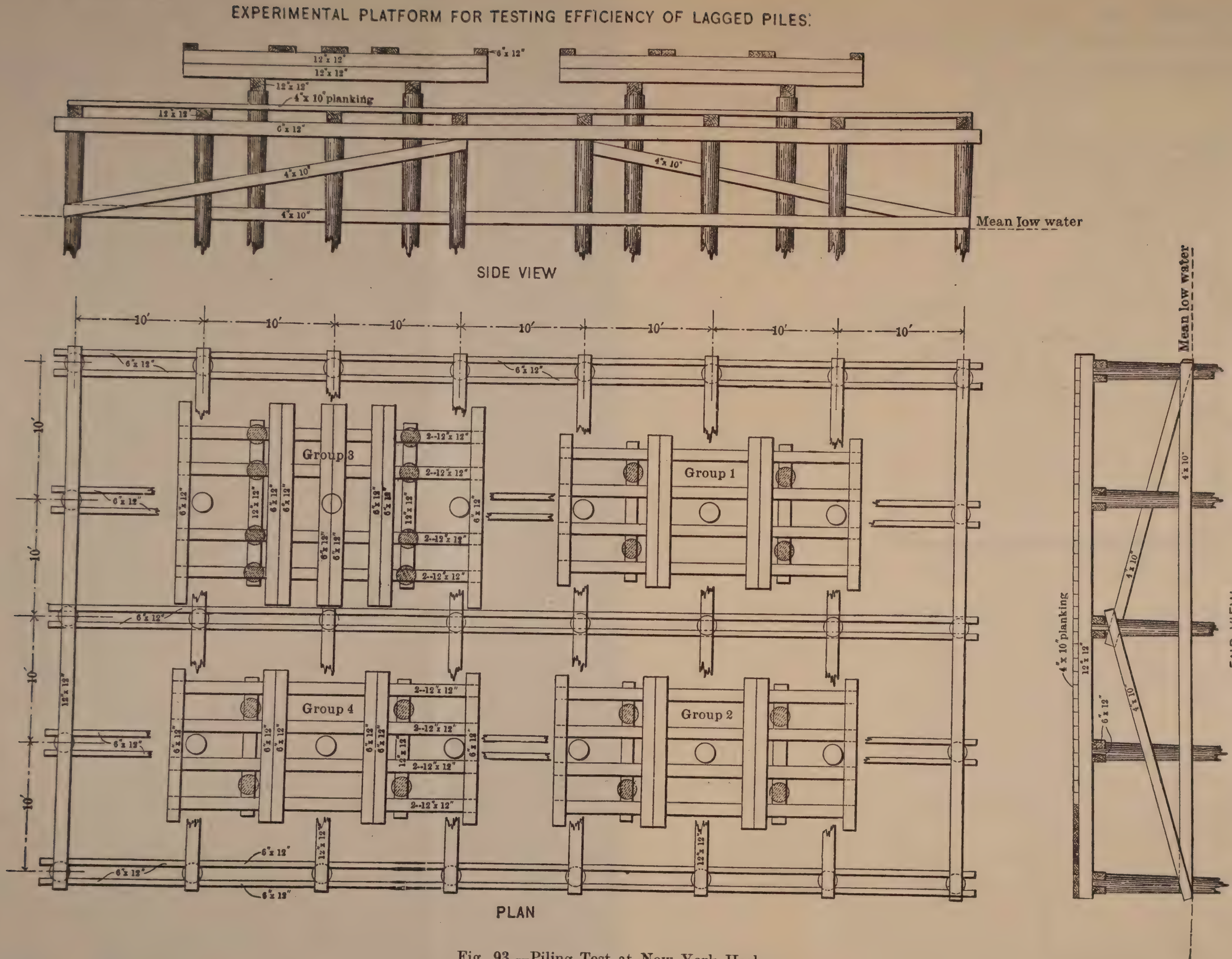


Fig. 93.—Piling Test at New York Harbour.

Group IV., piles lagged with two pieces of 5-inch by 6-inch lumber, and two pieces of 4-inch by 10-inch lumber, in lengths of 30 feet, this with the object of obtaining results for a different style of lagging.

The testing platforms were loaded with granite and concrete blocks eighteen days after the last pile had been driven, thus affording the mud an opportunity of consolidating round the piles. A boring, taken at the site of the platform, indicated mud of uniform character to a depth of 100 feet below mean low water-line. The consistency of the mud at the top was such as to admit of the piles sinking by their own weight through 10 or 15 feet when lowered gradually; a little further down, the mud attained the consistency of wet modelling clay. The depth of water was 22 feet below mean low water level, and the piles were driven by a hammer weighing about 3,000 lbs., having a uniform effective fall of about 8 feet.

The experiments were spread over a period of fifty-four days, when they came to an abrupt conclusion owing to the failure of the platform under the wash occasioned by the passage of a steamship. The last observations taken showed the maximum settlement of any test pile to be about $3\frac{1}{2}$ inches, and that the settlement of the working platform in its vicinity was $1\frac{3}{4}$ inches. It is to be noted that this latter settlement took place under no load beyond the weight of the piles and the timber upon them, and that, therefore, the maximum settlement under load of any test pile was practically only $1\frac{3}{4}$ inches. A settlement of this amount appears to be not uncommon in local piers formed of similar piles, even before the structure is finished, or has received any other load than its own weight.

Mr. Bensel, from various considerations indicated in his report, concludes that the ultimate bearing power in the unlagged piles of Group I. might be taken at 20 tons per pile, and that in the remaining groups of lagged piles the ultimate bearing powers would be 30, 20, and 30 tons respectively, per pile.

The table on p. 126 shows details of the observations made during the experiments. The tons are given in American units of 2,000 lbs. Roughly, their equivalent value in English units of 2,240 lbs. may be arrived at by deducting one-tenth. This modification applies also to the figures quoted in the preceding paragraph.

In the San Francisco Harbour Commissioners' Report for 1899 there is a record of a test pile 90 feet long loaded with 45 tons for 48 hours without any subsidence. At Fort Mason a test pile 88 feet long driven 53 feet into the mud penetrated 1 inch under the last blow of a 4,000-lb. hammer, dropping 15 feet. This pile was loaded with 48 tons of pig iron for 60 hours and no subsidence occurred.[1]

The following data [2] relating to piles driven at Portsmouth Dock Yard

[1] Report of the Engineers of the Federated Harbour Improvement Associations on San Francisco Harbour, 1908. The sectional area of the piles is not stated.

[2] *Min. Proc. Inst. C.E.*, vol. xliv., p. 204.

Pile Tests at New York Harbour.

Group.	Pile.	Diameter of point.	Diameter of butt.	Approximate weight of pile.	Total number of blows to drive.	Total penetration into mud.	Surface contact of pile with mud.	Average penetration for each of last five blows	Load supported during latter twenty-seven days of test.	Settlement under load as in preceding column.	Settlement in working platform during same period.	Remarks.
		Ins.	Ins.	Lbs.		Ft.	Sq. ft.	Ins.	Tons.	Ins.	Ins.	
I.	1	8	18	4200	26	48	139	8	18·3	$\frac{1}{8}$	...	Unlagged piles 5 ft. 6 ins. apart; in pairs 11 ft. 6 ins. apart.
	2	8	17	3900	19	49·6	134	$9\frac{1}{4}$	18·7	...	...	
	3	7	14	2700	13	50·9	127	$13\frac{1}{4}$	18·7	$\frac{1}{8}$	...	
	4	$6\frac{1}{2}$	17	3640	11	45·7	123	$14\frac{1}{4}$	18·7	...	...	
II.	1	6	17	3600	46	49·6	220	4	31·8	$\frac{13}{16}$	$\frac{5}{16}$	Piles spaced as above and lagged with four 5-in. × 6-in. pieces 30 ft. long.
	2	$6\frac{1}{2}$	15	2940	58	50·1	219	$3\frac{5}{8}$	31·8	$1\frac{9}{16}$	$\frac{5}{16}$	
	3	$7\frac{1}{2}$	16	3450	46	51·0	233	4	31·8	$\frac{11}{16}$	$\frac{5}{16}$	
	4	8	$16\frac{1}{2}$	3700	69	49·3	238	3	31·8	$\frac{13}{16}$	$\frac{5}{16}$	
III.	1	7	17	3700	70	50·7	247	$3\frac{1}{8}$	28·0	$1\frac{1}{16}$	$\frac{7}{8}$	Piles lagged as in Group II. and arranged in two lines of two pairs each, the space between each line being 11 ft. 6 ins.; between the pairs in each line, 5 ft 6 ins.; and between the piles in each pair, 2 ft. 8 ins.
	2	8	$17\frac{1}{2}$	4060	70	49·8	243	$3\frac{1}{8}$	28·0	$1\frac{1}{8}$	$\frac{7}{8}$	
	3	$7\frac{1}{2}$	$18\frac{1}{2}$	4370	64	49·0	241	3	28·0	$1\frac{1}{8}$	$\frac{7}{8}$	
	4	9	18	4450	80	49·9	250	$2\frac{5}{8}$	28·0	$1\frac{1}{16}$	$\frac{7}{8}$	
	5	6	22	5700	90	52·6	257	$2\frac{3}{8}$	28·0	$1\frac{9}{16}$	$\frac{7}{8}$	
	6	$7\frac{1}{2}$	21	5500	67	49·4	247	$2\frac{3}{4}$	28·0	$1\frac{11}{16}$	$\frac{7}{8}$	
	7	$5\frac{1}{2}$	22	5660	72	49·7	243	$2\frac{3}{4}$	28·0	$1\frac{1}{8}$	$\frac{7}{8}$	
	8	9	19	4870	83	48·8	245	$2\frac{1}{2}$	28·0	$1\frac{1}{2}$	$\frac{7}{8}$	
IV.	1	5	$16\frac{1}{2}$	3270	59	50·7	241	$3\frac{5}{8}$	34·6	$\frac{5}{8}$	$\frac{3}{4}$	Piles spaced as in Group II. and lagged with two pieces 5 ins. × 6 ins. and two pieces 4 ins. × 10 ins. all 30 ft. long.
	2	10	$18\frac{1}{2}$	4870	77	49·4	247	$2\frac{1}{2}$	34·6	$\frac{3}{8}$	$\frac{3}{4}$	
	3	$7\frac{1}{2}$	19	4600	44	47·1	231	$4\frac{3}{8}$	34·6	$1\frac{11}{16}$	$\frac{3}{4}$	
	4	7	$17\frac{1}{2}$	3890	65	47·8	233	$2\frac{7}{8}$	34·6	$1\frac{9}{16}$	$\frac{3}{4}$	

extension constitute a typical record of ordinary experience in driving piles into very firm ground. The ground consisted of a regularly stratified, argillaceous sand, containing perhaps half its bulk of pure clay. The beds were fine, some of them not being half an inch thick, wholly impervious to water across the stratification, and very slightly, if at all, pervious in the direction of stratification.

All the piles were of fir, 17 feet 4 inches long and $13\frac{3}{8}$ inches square. Six of them were driven by hand by five men with a monkey weighing 15 cwts., and six by steam with a monkey weighing 22 cwts. The maximum fall in the first case was $21\frac{3}{4}$ feet, and in the second case $14\frac{1}{4}$ feet.

Pile.	Time in Driving.	No. of Blows.	Maximum Fall.		Final Depression.	Total Penetration.			Remarks.
	Hrs.		Ft.	Ins.	Ins.	Ft.	Ins.		
1	14	228	19	0	$\frac{1}{4}$	12	9	Hand machine.	Piles in all cases except two driven to the stage of pronounced or incipient splitting. In cases 9 and 12, as much as 3 feet and 4 feet 6 inches respectively had to be cut off the top of the piles from this cause.
2	12	137	20	1	$\frac{1}{2}$	11	8		
3	$15\frac{1}{2}$	179	21	6	$\frac{5}{16}$	12	3		
4	15	169	21	10	$\frac{3}{8}$	12	4		
5	$18\frac{1}{4}$	270	21	9	$\frac{1}{4}$	13	11		
6	14	181	21	7	$\frac{1}{2}$	13	2		
7	$3\frac{1}{4}$	265	7	8	$\frac{1}{4}$	10	6	Steam machine.	
8	2	255	14	3	$\frac{3}{16}$	14	3		
9	$1\frac{1}{2}$	280	9	5	$\frac{1}{4}$	14	8		
10	$2\frac{1}{2}$	180	10	9	$\frac{1}{2}$	14	0		
11	2	228	10	8	$\frac{1}{8}$	13	9		
12	$3\frac{1}{4}$	130	9	10	$\frac{3}{8}$	11	0		

Formulæ professing to give the exact sustaining power of piles are numerous, radically different in form, and conflicting in results. They are to be found in all engineering pocket-books, and little advantage would be derived from quoting them here. Some are extremely complex, embodying elements which have little or nothing to do with the capabilities of a pile to sustain an imposed load. When all has been said, it must be evident that the true test of sustaining power is the resistance offered to the final blow. The length, weight, and modulus of elasticity of the pile are factors possessing no practical value, and a simple formula, linking up the weight of the ram and its fall with the resulting depression, should give all that is required. Major Saunders's formula is certainly based on these lines, but, unfortunately, it does not adapt itself to all cases. Thus, with a depression of $\frac{1}{10}$ inch under the last blow of a 2,000-lbs. ram falling 9 feet, the safe load becomes 270,000 lbs. To sustain such a load, a pile, in the author's estimation, should be not less than $15\frac{1}{2}$ inches square, and, by Rankine's rule, $16\frac{1}{2}$ inches. Any pile, therefore, of less dimensions would be incapable of supporting so heavy a load with reasonable regard to safety, whereas, as a matter of fact, many 11- and 12-inch piles have been driven to comply with the standard stated in the formula. The values given by the formulæ of other authorities for similar conditions are as follows:—Haswell, 30,000 to 60,000 lbs.; Weisbach, 26,800 to 28,000 lbs.; Wellington, 32,727 lbs.; Trautwine, 32,460 to 97,380 lbs., the range in each case being due to the limits in the coefficient or factor of safety, which must always remain a matter of conjecture and arbitrary selection. The multiples, in fact, lie anywhere between $\frac{1}{2}$ and $\frac{1}{12}$.

Another and no less essential point to be noted is that the values obtained by these formulæ relate solely to loads imposed upon piles which are completely embedded in the ground. In so far, therefore, as a pile acts merely as a foundation for a pier column, the foregoing estimates of its resistance to pressure are strictly and legitimately applicable. But when a pile is only partially embedded in the ground, the calculations for its stability are of a dual nature: first, as a pile pure and simple up to the surface of the ground, and secondly, above the ground level, as a column or strut.

This aspect of the case calls for careful consideration, because a framework wharf, or pier, may fail through the flexure of its vertical members as much as through the subsidence of their bases. The longer the unsupported length, the less becomes the permissible load. And it follows, as an obvious corollary, that cross and diagonal bracing should be introduced from the lowest level at which it becomes practicable.

Failure by flexure involves an investigation of the relative values of the resistance of a material to tension and compression. Within the limits of the present treatise, it is not feasible to enter into all the details of so complex a problem. Neither in the present connection is it altogether desirable. Gordon's well-known formula will be found sufficient for determining in a general, practical way the limiting load on columns, whether in the sea

or ashore, and any further information on the subject should be sought in works dealing specially with columns and structural work generally.

Gordon's formula for the determination of the limiting loads on long columns or struts may be expressed as follows :—

$$p = \frac{f}{1 + a\frac{l^2}{d^2}},$$

where p = ultimate load per square inch of sectional area ;
f = compressive stress per square inch of the material ;
$\frac{l}{d}$ = ratio of length of column to its diameter, or to its least dimension in cross-section ;
a = coefficient given in table below.

The ultimate compressive stress (f) may be taken as follows, according to the material of which the strut is composed :—

Timber, . . .	2 to 4	tons per	square inch.
Wrought iron, . .	16	,,	,,
Mild steel, . .	30	,,	,,
Cast iron, . . .	40	,,	,,
Concrete (4 to 1), .	2	,,	,,
,, (8 to 1), .	1	,,	,,

The values of the coefficient a are given in the subjoined table.

COEFFICIENTS IN GORDON'S FORMULA.

Material.	Cross Section.	Values of a.		
		Both Ends Rounded.	Both Ends Fixed.	One End Rounded and Fixed.
Timber, . .	Rectangular or circular, .	$\frac{4}{250}$	$\frac{1}{250}$	$\frac{1}{100}$
Wrought iron, .	Rectangular, . . } Circular (solid or hollow), }	$\frac{4}{2500}$	$\frac{1}{2500}$	$\frac{1}{1000}$
,, .	L T + ⊏ ⊜ I ⊔ .	$\frac{4}{900}$	$\frac{1}{900}$	$\frac{1}{360}$
Cast iron, . .	Circular (solid), . .	$\frac{1}{100}$	$\frac{1}{400}$	$\frac{1}{160}$
,, . .	,, (hollow), . .	$\frac{1}{200}$	$\frac{1}{800}$	$\frac{1}{320}$
,, . .	Rectangular, . . .	$\frac{3}{400}$	$\frac{3}{1600}$	$\frac{3}{640}$
,, . .	Cross-shaped, . . .	$\frac{3}{200}$	$\frac{3}{800}$	$\frac{3}{320}$
Reinforced concrete, {	Circular (solid), . . .	$\frac{1}{600}$	$\frac{1}{2400}$	$\frac{1}{960}$
	Rectangular (solid), . .	$\frac{1}{784}$	$\frac{1}{3136}$	$\frac{1}{1254}$

In the case of compound columns of concrete and steel (reinforced concrete), it is necessary to find the equivalent sectional area in terms of one material and make the ensuing calculations on that basis. Thus, if ρ be the ratio of the coefficient of elasticity of steel to that of concrete—that is, if

$\rho = \frac{E_s}{E_c}$, then an area A of steel is equivalent in resistance to an area ρ A of concrete. If A is the area of a reinforced concrete section (including the area of the steel reinforcement), and A_s is the area of the steel, then the equivalent section A_c in simple concrete will be

$$A_c = A + (\rho - 1) A_s.$$

The strength of a reinforced concrete pile can, therefore, be determined by treating it as a simple concrete pile of augmented area, the equivalent area being determined as above. The value of *a* in the table corresponds to this method of treatment. The value of ρ may be taken as 15.

CHAPTER VI.

STONE: NATURAL AND ARTIFICIAL.

Stone Supplies—Qualities desirable—Density and Hardness—Weight of Stone—Obtainment—Mine Firing—Drilling Operations—Implements—Charging—Tamping—Firing—Fuses and Detonators—Seam Firing—Wedging—Blasting Agents—Description of Quarrying Operations for Breakwaters at Goodwick, Alderney, and Holyhead—Concrete—Its Ingredients—Their Qualities and Proportions—Sea-water in its relationship to Concrete—Model Specification for Concrete in Maritime Works—Japanese Standards.

Stone.

Natural Stone.—One of the most important considerations in connection with the construction of a breakwater is the supply of stone. Even in the case of those breakwaters which consist mainly of concrete blocks, it is eminently desirable, from an economical point of view, to pack the concrete with as many stone burrs, plums, or displacers, as possible. And in mound breakwaters a plentiful supply of rubble is obviously a paramount requirement.

The matter opens out into two branches. First, there is the quality of the stone, and secondly, the cost of obtaining it. The former question involves a consideration of physical characteristics and chemical qualities; the second, that of the proximity of a suitable quarry and the means of transport.

Quality.—In regard to physical characteristics, there are two features of pre-eminent importance—density and hardness. *Density*, or high specific gravity, is essential, because, when immersed in water, a stone loses a very considerable part of its effective weight; and when the sea is in motion, its stability as an inert mass is thereby reduced to a very great extent. Furthermore, if, compared with its weight, the stone possess a very large bulk, it presents a correspondingly large surface to wave action, thus increasing the scope or field of the disturbing force. These two factors of volume and weight must, therefore, be taken into joint consideration; they show that the smaller the surface area of a stone and the greater its unit weight, the less likelihood there is of disturbance. In other words, the higher the specific gravity, the greater the stability.

A concrete example will perhaps render this fact clearer. Take two blocks of stone of the same size—say exact cubes, each containing 1 cubic yard—but with specific gravities, represented in one case by 3 and in the other by 2. In air, the weights are 5,184 lbs. and 3,456 lbs. respectively. In sea-water, the weights (after deducting the weight of the volume of water, which is the same for both) are 3,456 lbs. and 1,728 lbs.—a ratio of 2 to 1,

representing an increase of 33 per cent. As the exposure to wave-stroke is the same in both cases, it is obvious that, when immersed, the stability of one block has relatively increased from half as much again to twice that of the other.

The second point is *hardness*, or durability. A good stone in this respect is one which is dense, compact, impervious, and free from all susceptibility to disintegration. In maritime situations, stones are subjected to much wear and friction—certainly more so than on land. The swell of the sea keeps those of small size in a state of continual agitation, rolling them over one another and chafing them until they assume that smooth spherical or ellipsoidal form which is so characteristic of pebbles along the beach. Moreover, in stormy weather, shingle, shells, and gravel are taken up by the waves and dashed with tremendous force against any surface upon which the waves happen to break. The effect of continual impact of this kind is to wear away even the hardest masonry. Wave action is supplemented by that of the wind, which blows sand in great volumes with the severity of a sand-blast. The cumulative results of abrasion and attrition are to be observed on any rocky coast, where towering cliffs stand honeycombed and fretted into fantastic shapes, while the strand is strewn with the comminuted fragments of quondam boulders.

The chemical qualities of a stone are not perhaps of such striking importance as its physical characteristics, but they are nevertheless deserving of consideration. The acidity and salinity of sea-water may, and often does, bring about molecular changes in minerals containing soluble salts. Certain compounds of lime are decomposed and softened by sea-water, and they also give rise to the formation of other compounds which tend to destroy the cohesion of the material of which they are ingredients, by producing cracks and fissures. Caustic lime and caustic magnesia, which are to be found in inferior and imperfectly made artificial stone or concrete—more rarely in natural stone—are causes of disintegration by reason of their expansion under hydration, and also on account of their solubility. Still, on the whole, the chemical aspect of the question assumes a secondary importance, because those rocks which come under the category of minerals available for marine purposes, on account of their physical properties, are mostly, if not altogether, free from unstable constituents. The only exception, perhaps, is granite, which is a composition of three minerals—quartz, felspar, and mica, in a state of physical, not chemical, incorporation. Of these, the quartz is durable beyond cavil—it is practically indestructible; but certain varieties of felspar are liable to decomposition, and the mica is always more or less easily disintegrated. Nevertheless, granites, as a class, have gained a high reputation for strength and permanence, and it is only in very inferior qualities that the imperfections just mentioned manifest themselves, or where any appreciable deterioration is produced by natural agencies.

The heaviest and most durable varieties of stone are, generally speaking, those of igneous origin, such as basalts, granites and traps, and metamorphic

rocks such as quartzite. Many of the harder sedimentary rocks, though suitable in other respects, are unfortunately subject to the depredations of two troublesome molluscs, the *Pholas dactylus* and the *Saxicava*, both of which attack limestone and sandstone. Limestone blocks at Plymouth breakwater have had to be replaced by granite blocks on account of the ravages of the *Pholas*, which has already been mentioned in connection with its attacks on timber structures. Boring its holes in close proximity to one another, it honeycombs masonry work until it brings about its destruction.

The weights and specific gravities of stone suitable for maritime purposes are somewhat as follows. It will be understood, of course, that there is often a considerable range of weight in material of the same class, according to locality, owing to variations in composition and texture.

WEIGHT AND STRENGTH OF STONE.

	Weight in Lbs. per Cub. Ft.	Crushing Load in Lbs. per Sq. In.	Specific Gravity.
Granites, . . .	160 to 190	8,000 to 14,000	2·5 to 2·97
Basalts and Traps, .	170 to 190	8,000 to 16,000	2·65 to 2·97
Limestones, . . .	130 to 170	3,000 to 9,000	2·03 to 2·65
Sandstones, . . .	150 to 170	2,000 to 8,000	2·34 to 2·65

Granite has been used in the construction of two notable breakwaters in this country—those of Plymouth and Portland. The stone used at Plymouth came from the quarries of Colcerrow and Roughtor in Cornwall and Pewtor in Devonshire. Penryn, near Falmouth, in Cornwall, supplied stone to Portland, where a large quantity of the local limestone was also used. Holyhead breakwater was built of Anglesea stone, which, nominally a granite, is really a quartzite. Alderney breakwater consists mainly of the native Mannez stone, a sandstone grit of such extraordinary hardness as to exceed that of the neighbouring Guernsey granite.

The limestone rubble for Gibraltar harbour works was obtained from local quarries at Catalan Bay and Europa Point ; most of the granite came from Cornwall, some from Italy, and a little from Norway.

Obtainment.—Next to the selection of a stone comes the question of the facilities for its obtainment and the cost of conveyance. Certain breakwaters have been so fortunate as to be located in the immediate neighbourhood of a suitable quarry. In other cases stone has had to be transported from some distance. Generally speaking, upon a rocky coast stone is likely to be fairly plentiful and cheaply procurable ; on a sandy shore, where stone is not so accessible, other forms of construction, such as fascine work, may commend themselves to preference on economical grounds.

Quarrying.—The art of quarrying is one which is often applied to special purposes : some quarries being mainly worked for building blocks, and

others almost entirely for setts and road metalling. Obviously, neither of these departments claim any attention here. Stone which is required for breakwater purposes is of an intermediate character—not so small as for macadam, nor so regular as for architectural work. The rubble which is desirable for maritime undertakings is of varying size, and, in fact, is such as results more or less naturally from the simple blasting of rock. Except for copings and string courses, no dressing is required, and the main bulk of the work may be executed in blocks of irregular size and shape. In order to obtain these blocks to fairly large dimensions, some discrimination has to be exercised both in regard to the manner of boring the holes for blasting purposes and the nature and amount of the charges employed.

Much, of course, depends upon the disposition of the working face of the quarry and its relationship to the strike and dip of the strata. Natural joints and beds should obviously be taken advantage of to the fullest extent. These features are most irregular and uncertain in the igneous rocks, and, therefore, call for the aid of some skill and experience in their utilisation.

When blasting operations are projected on a large scale, the system of **mine firing** is adopted, and headings are driven in from a vertical face, or shafts are sunk from the top—the relative economy of these methods being dependent on the height of the quarry escarpment. Drainage and ventilation are more readily assured by the use of headings. In this case galleries are formed of the smallest possible sectional area consistent with the working space required for a man in each; they are arranged zig-zag in direction or with one or more abrupt turns, and they terminate in chambers which are filled with explosives. Shafts, on the other hand, are straight and vertical.

Mine firing, which produces huge downfalls of stone—ranging, in many instances, from 100,000 tons to 500,000 tons—results in the dislodgment of so many and such enormous masses of rock that these last have to be again broken up into serviceable sizes by means of smaller charges. The method, therefore, does not altogether obviate the alternative system of small-charge firing, which, in less extensive operations, is more generally adopted.

Drilling Operations.—For the purpose of boring the necessary holes, either to receive the blasting charge proper or as a preliminary in the formation of a shaft or heading, various kinds of drilling instruments are employed, including the jumper, the hand-drill, and the machine drill. Of these, the two former involve manual labour; the last is mechanical and automatic. Where the work is sufficiently extensive to justify the initial cost of installation, there can be little doubt as to the superior economy and efficiency of machine drills. They can be worked much more accurately and with greater ease and convenience, there being situations where the application of hand-drilling would prove awkward, tedious, and slow.

The *Jumper* is an implement worked by one or several men. It consists of a long heavy bar of steel, sometimes circular or cruciform in section, but generally octagonal. The length varies from 6 to 8 feet, and, though not

commonly the case, the bar is sometimes thickened in the middle in order to give increased momentum to the blow. In drilling a vertical hole, the jumper is lifted and allowed to fall, its uprightness being maintained throughout. It is caught at each rebound and raised again, being given, at the same time, a slight turn. For horizontal work, the drill is swayed backwards and forwards and slowly rotated as before. One drawback of the jumper is its liability to deflection from its assigned direction if it happens to come across a vein of harder material. Guidance is practically absent at the moment of impact.

The *Hand-drill* is a short steel bar of octagonal section, manipulated either by one man, who holds the bar with one hand while he strikes it on the head with a hammer held in the other, or by two, or even three men, one of whom acts as holder and the others as strikers. The command over a hand-drill is more effective in maintaining its alignment than it is in the case of a jumper.

The limiting amount of useful penetration by the hand-drill is about 2 feet, and it is chiefly used for making the short plug-holes, some few inches in depth, which enable large blocks to be split up into smaller pieces. The

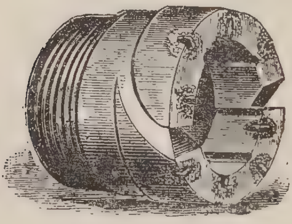


Fig. 94.—Core-bit or Cutting Edge of Rotary Drill, set with Diamonds.

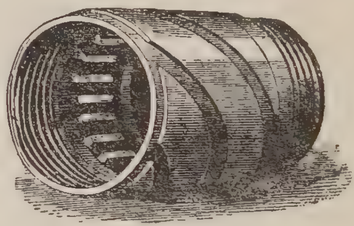


Fig. 95.—Core Lifter.

jumper may be effectively used for holes of from 3 to 4 feet in depth. The rate of progress in either case depends, of course, mainly on the hardness of the rock, and, in the second place, on the diameter of the hole, but it may be taken, on an average, at from 1 foot to 5 feet per hour ; the former rate for holes of 2 inches diameter in granite, and the latter for $1\frac{1}{2}$-inch holes in limestone.

At the Kirkmabreck granite quarry, the following rates of work obtained —viz., three men could bore about 7 feet per day of $2\frac{1}{2}$-inch hole, and $8\frac{1}{2}$ feet per day of 2-inch hole. Of plug-holes 9 inches deep, three men could drive 24 feet per day to $1\frac{1}{2}$ inches diameter, and 32 feet per day of holes $\frac{7}{8}$ inch diameter and 4 inches deep. One man alone could do 14 feet per day of $\frac{5}{8}$-inch plug-holes, 3 inches deep.

Machine drills are either rotary or percussive in action, and are actuated variously by steam, compressed air, water under pressure, and electricity.

Purely *rotary drills* generally take the form of a tube with an annular cutting edge, formed either with hardened steel teeth or with a row of diamonds. In the Brandt drill, steel teeth are forced against the surface of

the rock under enormous hydraulic pressure, while the tube makes from five to eight revolutions per minute. In the ordinary diamond drill, the periphery of the "core-bit," as it is termed, has a number of diamonds embedded in it, and rotation is much more rapidly performed—from 200 to 400 revolutions per minute. The core, which results from the action of the tube, is subsequently broken off and withdrawn by a "core-lifter," which forms part of the internal mechanism of the drill.

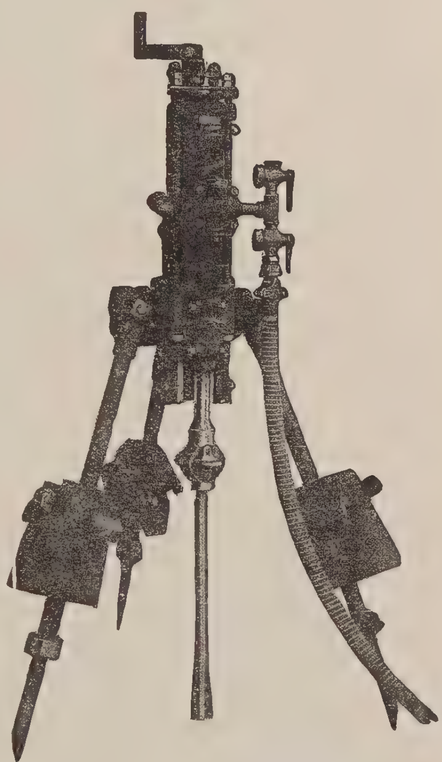


Fig. 96.—Ingersoll Percussive Drill.

The annular form of such drills lends itself to the supply of water to the point of incision, an adjunct which is of decided advantage in all forms of drilling, whether by hand or mechanism. In this respect hydraulic motive power may serve a double purpose, the waste water from the pressure cylinder acting also as a lubricant and dust preventer.

Percussion drills, which also have a subsidiary rotary movement, conform to the principle of the manual drill in that they are driven forcibly against the rock by steam or other pressure. The essential parts are a cylinder and piston, the latter of which receives the pressure alternately on each of its faces and acts as a combined hammer and drill, or perhaps more closely resembles the jumper. The drill rod proper is solid throughout and attached to the end of the piston. It is provided with a cutting edge or bit, of I, X,[1]

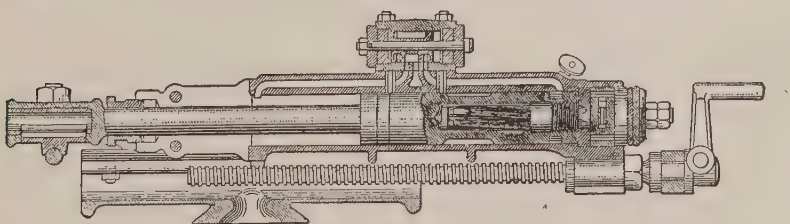


Fig. 97.—Section of Ingersoll Drill.

[1] An X shape is preferable to an exact cross, as it affords less likelihood of the same grooves being struck repeatedly.

or Z shape. The bit requires sharpening every 2 to 4 feet of penetration. The pressure employed is about 60 to 70 lbs. per square inch. About 300 blows are delivered per minute, and the rate of progress ranges from 3 to 10 feet per hour when the diameter of the hole lies between 1 and 2 inches. One man suffices to operate a machine, which may comprise several drills, but two or three men are required to transport it, and two are generally in attendance. In granite, two men working a steam drill can do about three times the amount of work which would be done by hand in the same time.

Fig. 98.—Drill Steels.

The cutting edges, or "bits," of percussive drills, the widths of which range downwards from 4, or even 5 inches in mechanical drills and from 2 inches in manual drills, are slightly wider than the shanks of the bars on which they are worked, in order to ensure the necessary clearance in driving. For the same reason, the diameter of the drill is diminished as the depth attained is increased, at the rate of about $\frac{1}{16}$ inch every 18 inches drilled by hand, and about $\frac{1}{8}$ inch every 2 feet drilled by machine. Thus a 20-foot hole commencing with a diameter of $3\frac{1}{4}$ inches at the top would become reduced to 2 inches by the time the bottom was reached.

While the steam-driven percussive drill is of great value for heavy work and deep borings, increasing scope has been found for the use and development of portable hand drills, driven by compressed air. These drills are designed mainly for holes of from 8 to 10 feet in depth, and they are operated under air pressure of from 60 to 100 lbs. per square inch. The drill steel is separate from the piston, which acts as a hammer on the head of the former, much in the same way as a hand drill is operated, the internal mechanism providing for the necessary rotation of the cutter. It is not usual to apply such drills to deep holes, but, as an instance of what can be done in exceptional circumstances, a drill of the jack-hammer type has proved capable of boring a 21-foot hole in hard pennant stone, starting at $2\frac{1}{2}$ inches diameter, in just an hour's time. Naturally, however, performances vary with the character of the stone and the intensity of the pressure.

A drawback to the use of steam is the amount of condensation which takes place in the piping when it is of any considerable length—say, exceeding

150 yards or so. Furthermore, the exhaust from the drill is often very inconvenient, obscuring the work and rendering it difficult and troublesome to the operator. On the other hand, where the drilling is relatively small in amount and at places widely apart, the installation of an air compressor plant would not be economically practicable. In such cases the steam-driven unit is alone suitable.

Charging.—After driving has been completed to the required extent, the hole is cleared of débris and moisture prior to the insertion of the charge. The amount of the charge is calculated on the same principle as that which underlies the preparation of the borehole—viz., that the explosive acts in the direction of the line of least resistance, or along the shortest route from the charge to the nearest open face; the hole should be considerably longer than this distance, at least twice as long. On the basis stated, we have the following formula :—

$$\text{Charge in lbs.} = (\text{line of least resistance in feet})^3 \times \text{coefficient},$$

in which the coefficient depends upon the nature of the rock and of the charge, being only definitely determinable by actual experience. For ordinary blasting powder in granite it is approximately ·04, and in softer stone ·03. For higher explosives it will be less, in proportion to their specific power. Thus for dynamite the coefficient becomes ·005 or ·004. Gunpowder exerts, according to its precise composition, an explosive force of from 18 to 40 tons per square inch. For blasting purposes only the lower power is used, and a cubic yard of quarry rock ordinarily requires a charge of from $\frac{1}{4}$ lb. to 2 lbs. according to its nature and position; in tunnels and shafts as much as 6 lbs. per cubic yard has been used.

Blasting powder may be deposited in the borehole in bulk, but high explosives are usually made up in the form of cartridges, and the diameter of these is arranged so as to fit the hole exactly. The charge will consist of as many cartridges as may be considered necessary, carefully pressed into contact with one another by means of a blunt-ended wooden tamping-rod. The topmost cartridge is fitted with the cap, or detonator, required to produce explosion.

Tamping.—The next step after charging, prior to explosion, is tamping. This consists in packing the hole with granular or plastic material, so as to completely confine the charge and its gaseous products. For powder, dry, tough clay and powdered brick make good tamping material. By reason of its rapid action, dynamite does not require tamping to such an extent as is necessary for slower explosives. Even water will serve the purpose in deep vertical holes. Mud, sand, brickdust, and clay are all used in connection with high explosives. Care must be taken in pressing home the first portion of a tamp, so as to avoid prematurely exploding the cap. Only a blunt wooden tamping-rod should be used.

Firing.—There is a difference of procedure in regard to the circumstances

attending detonation. Low explosives, such as gunpowder, expand progressively by combustion, the gases accumulating until the resistance to expansion gives way. High explosives, on the other hand, act instantaneously, but they require a sharp initial explosion to develop their action fully, and this is provided by means of the detonator.

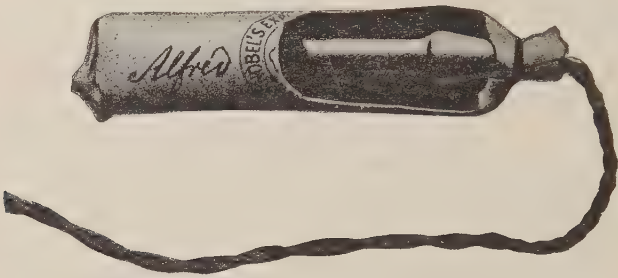


Fig. 99.—Electric Detonator Fuse inserted in Cartridge.

The *detonator* in general use is a small, solid-drawn copper tube, closed at one end and partly filled with an explosive compound (fulminate of mercury and chlorate of potash in varying proportions) which is capable of producing intense local force and heat. The detonator itself may be exploded either by means of a combustible fuse or by the electric current.

The ordinary (safety) combustible fuse burns at the rate of 2 to 3 feet per

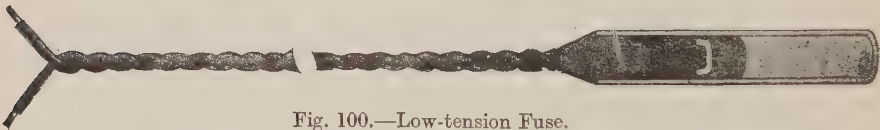


Fig. 100.—Low-tension Fuse.

minute. There is also a "lightning" fuse for simultaneous firing, burning at the rate of 150 feet per second.

Electrical discharge, of course, takes place instantaneously. There are two methods of firing. One, called the *low tension* (fig. 100), consists in sending an electric current through a thin platinum wire contained in the detonator, whereby it is made red-hot and so ignites the inflammable material

Fig. 101.—High-tension Fuse.

in which it is incased. In *high tension* fuses (fig. 101), the wire terminals are not connected, but a spark is passed between them, effecting the same result.

Seam Firing.—It has already been remarked that the efficacy of a blast depends to a very large extent upon the judicious selection of the site and direction of the bore-holes. The natural faults to be found in rock manifestly

lend themselves to an economical disposition of the disruptive forces, and it is often a wise plan to try the effect of a small preliminary charge by way of ascertaining to what extent any latent lines of weakness are developed. Thus, in a quarry at Penmaenmawr, in North Wales, where the rock is a hard, compact trap, it is the practice to bore a hole, say 20 to 23 feet deep, and charge it with only 5 lbs. or so of powder. On explosion, certain cracks are produced, the traces of which are followed and enlarged by additional holes until a final charge can be suitably placed for bringing down the whole mass. Similarly, at Goodwick, Pembrokeshire, a hole 20 feet deep and $2\frac{3}{4}$ inches in diameter at its extremity would be sprung several times by means of single gelignite cartridges of $1\frac{1}{4}$ inches diameter, producing a pear-shaped cavity capable of receiving the larger quantity—50 or 60 lbs.—required for complete dislocation.

Another method adopted in a granite quarry at Kirkmabreck, Kirkcudbrightshire, where the seams are fairly regular, extending along the face-lines for some distance, was to open out a seam by means of plugs and wedge, until it was wide enough to admit of the lodgment of a charge of powder. A chamber for the charge was prepared by shaping a couple of boards to fit the seam and setting them temporarily a couple of feet apart. Tamping was tightly rammed against the outsides of the boards, which were then withdrawn. The method of charging was to insert the fuse after the deposition of a third of the powder, the remainder being added on top and covered with a layer of lightly pressed hay followed by tamping above. The first foot of tamping was lightly rammed, the rest compactly.

Wedging.—Where the rock is of good quality and is required in the form of large, sound blocks for ashlar work in copings, facings, and string courses, a system of obtainment by wedging is adopted in preference to that of blasting, which may produce unsuspected planes of weakness as well as undesirable cracking and cleavage. In this case, a series of plug-holes, about $1\frac{1}{2}$ inches in diameter and 9 inches deep, are driven along the line of some natural joint. Plugs and feathers are inserted into these holes, and driven by a succession of blows from a 26-lb. hammer until the seam has been sprung to an extent admitting of the use of plates and wedges. These last are driven simultaneously, and as the fissure produced widens out, it is kept well packed with hand rubble. When the block is detached and ready to come away, an iron dog is attached to the stone, from which a chain passes to a crane, the tension of which, aided by men with crowbars and levers acting directly upon the block, causes it to become completely dislodged. It can then be converted into convenient sizes by plugging as before, and be dressed to requirements.

When the height of the quarry face is considerable, it is desirable to place planks and pieces of old timber upon the ledges of rock and upon the floor of the quarry, to avoid undue breakage of blocks falling from the upper layers.

Blasting Agents.—The number of explosives available for working pur-

poses in a quarry is legion. From a practical point of view, despite the dissimilarity of their ingredients and methods of production, they fall into two classes—viz., (1) those in which a high local intensity is produced, causing much shattering and splintering into small fragments, and (2) those in which the expansive power is more widely and less violently exerted, resulting in disruption and dislocation rather than shattering. The first class is represented by dynamite, the second by gunpowder.

The basis of dynamite is *nitro-glycerine*, which forms a number of compounds possessed of similar attributes, but varying in power. Nitro-glycerine is a fluid combination of glycerine and of nitric and sulphuric acids. The majority of its various combinations, therefore, have a plastic, gelatinous nature, but the first to be noticed below has not this characteristic.

Dynamite consists of nitro-glycerine with the addition of a granular absorbent, which may either be an inert substance or, in itself, an explosive. The material more specially employed is a silicious infusorial earth occurring in Hanover, and called "kieselguhr." Commonly, the proportions are 75 parts, by weight, of nitro-glycerine to 25 parts of earth. If cartridges remain immersed in water for any length of time, the nitro-glycerine exudes and the charge deteriorates. Moreover, the substance is affected by changes in temperature and freezes at a higher temperature than the freezing point of water, so that some trouble is incurred in winter-time in thawing cartridges, the operation requiring much care and circumspection. The effects of firing are, as stated above, an intense rapidity of explosion producing extreme local shattering.

Other combinations of the same character are :—

Blasting Gelatine, containing 93 per cent. of nitro-glycerine and 7 per cent. of nitro-cotton. This is one of the most powerful blasting agents known at the present day. It is also very little, if at all, affected by immersion in water.

Gelatine Dynamite, somewhat inferior in strength to the foregoing, is a compound of nitro-glycerine, nitro-cellulose, wood meal, and nitrate of potash.

Gelignite contains nitro-glycerine, nitro-cotton, nitrate of potash, and wood meal. It is rather more powerful than ordinary dynamite.

Forcite is a mixture of nitro-glycerine with cellulose, the latter being gelatinised by heating in water under considerable pressure. Nitrated cellulose is also used in admixture with oxidising salts.

Gun cotton, which is cotton dipped into a mixture of nitric and sulphuric acids and itself an explosive, gives rise to the following, amongst other, compounds :—

Tonite is finely divided, or macerated, gun cotton, combined with an equal weight of nitrate of baryta. There are two varieties—the white and the black. The former is very shattering in its action, and is, therefore, chiefly applicable to the breaking up of extremely hard stone, such as quartz. *Black*

To face p. 141.]

Fig. 102.—Quarrying for Fishguard (Goodwick) Breakwater. The Cliffs before Blasting.

Fig. 103.—A Mine Explosion at Fishguard.

tonite, containing a larger proportion of baryta and some charcoal, is more disruptive.

Chlorate of potash forms the basis of two well-known explosives—viz., *Rack-a-rock* and *Cheddite*. The former consists of compressed cartridges of chlorate of potash, impregnated with dead oil, either alone, or in conjunction with bisulphide of carbon, or mixed with nitro-benzol. Cheddite, an admirable product of more recent date, contains chlorate of potash, naphthaline, and castor oil.

It is needless to extend the list further. There are many other excellent explosives on the market, and fresh compositions are continually being evolved, each with its own special advantages. But while many of them are characterised by extremely high power, resulting in the production of almost incredible downfalls of rock, yet in ordinary quarrying operations where, as has been pointed out, intense local effect is by no means sought after, it is probable that in the majority of cases blasting powder is every whit as serviceable, and certainly more economical.

Gunpowder, the earliest of explosives, is a mixture of saltpetre, sulphur, and charcoal, in proportions ranging between 6 : 1 : 1 and 15 : 3 : 2. These are the proportions used for service powder for military purposes. Blasting powder is distinguished from gunpowder, properly so-called, in that it contains rather less saltpetre and that it is not manufactured with the same particular selection of material and delicacy of treatment. The effective power is, therefore, lower.

Quarrying for Goodwick Breakwater.—The following particulars relating to the quarrying of stone for the breakwater in Pembrokeshire, forming a protection to the Fishguard terminus of the Fishguard, Rosslare (Great Western) route to Ireland, have been compiled from information kindly supplied by Mr. G. Lambert Gibson, the engineer in charge.

When the works were begun in the year 1896, they were carried out tentatively with a small outfit of plant, but with a considerable body of men. The start was a difficult one, the men having to attack the face of precipitous cliffs of an intensely hard and vitreous texture, rising from the sea to heights of one and two hundred feet. The boring of the rock to receive explosives was done entirely by hand; and, owing to the want of foothold, the men had often to be slung by ropes from the top of the cliff. After six years of somewhat slow progress in this manner, more vigorous measures were decided upon. A complete installation of compressed air drilling-plant was put down, and the work of boring was let to a firm of contractors.

Under the new system, both single firing and mine blasting operations were carried on. In the first case, holes 20 feet deep and $2\frac{1}{2}$ inches in diameter were charged with 20 to 50 lbs. of gelignite. In the second case, the method adopted where the cliff was lofty and the rock exceptionally hard, a tunnel some 40 feet long was driven square into the face of the cliff, with two branches, each also about 40 feet in length, right and left of it, the com-

bined galleries taking the shape of the letter T. At the ends of the cross tunnels small chambers were formed, within which were placed a charge of, usually, 7 tons of gunpowder in two boxes; the tunnels were then built up and the charges fired by electricity. Very little noise or shock to the neighbourhood is said to have been caused by the firing, although as much as 113,000 tons of rock have been dislocated by a 7-ton charge. The yield, however, varied considerably, according to the nature of the rock at various places along the half mile of quarry face, and, in some cases, 9 tons of gun-powder were required to produce a fall of 70,000 tons of rock.

Generally speaking, it was found that where a drill could penetrate 15 feet per day—the average depth of the holes—the cost of single hole-firing was equal to the cost of mine-firing with a working face of 120 feet. In other words, when the height of quarry face exceeded 120 feet, or when the drills failed to accomplish 15 feet per drill per day, mine-firing proved the more economical method.

The rock, having been blasted, was loaded into waggons. Stones from 3 to 15 tons weight were tipped on the sea side of the breakwater; those from 1 cwt. to 3 tons on the harbour side. Stones of 1 cwt. and less were sent to a ballast crusher for use in the concrete blockwork.

The rock-getting and depositing plant consisted of one 120 H.P. air-compressor engine, nine 8 H.P. Ingersoll rock drills, five locomotives, fifteen steam cranes of powers ranging from $1\frac{1}{2}$ to 15 tons and 175 waggons.

The quantity of rock dealt with amounted in all to about two million tons.

Quarrying for Alderney Breakwater.[1]—The stone of which the greater part of the breakwater is built is a local stone obtained from the Mannez quarries, and, although a sandstone grit, considerably harder than granite. The quarries were situated a couple of miles away from the site of the breakwater, and had a working face 75 feet high. The hardness of the stone may be gauged from the fact that where one jumper sufficed to bore a hole in granite, two were required for the Mannez stone.

The mode of quarrying was as follows:—Shot holes from 6 to 8 feet deep were drilled at the toe of the quarry face, charged, and exploded. When the rock was sufficiently undermined in this manner, a deep hole was drilled down from the top of the quarry to a shelf or bed, of which there were several, inclined at an angle of from 25° to 45° to the face. This hole was charged with powder, lightly tamped, and exploded several times, till a crack was made longitudinally; then, into the crack, large grained powder was poured and exploded, bringing down a considerable mass of rock. By this means the stone was not too violently shaken, and good face stones were obtained. Many of these were 9 feet long by 3 feet 3 inches thick and 15 feet on the bed. The cost of dressing them into shape came to 6d. per cubic foot. The

[1] Vide *Min. Proc. Inst. C.E.*, vol. xxxvii., p. 86.

[*To face p.* 142.

Fig. 104.—Quarrying for Fishguard Harbour.
The Cliffs after Blasting.

stones selected for rubble hearting weighed from 3 to 15 tons. The greatest quantity of material conveyed to the bank in one day was 3,000 tons. Seventy-four tons of powder were consumed per annum.

Quarrying for Holyhead Breakwater.[1]—The stone—a quartz rock—was obtained from an adjoining hill known as Holyhead Mountain, and the quarries were distant rather less than a mile from the commencement of the work.

At the outset, quarrying operations were carried on by a system of single hole-firing, but, although many hands were employed, the output proved insufficient for requirements. Blasting on a much larger scale was then resorted to, by sinking shafts and driving headings or driftings to receive large quantities of powder.

The first large mines were in shafts about 6 feet by 4 feet, sunk from the top and of varying depths, according to the height of the face; but when the quarries had been more opened and the face got very high, sometimes the top only was prepared for blasting by shafts, and the bottom by headings of the same size, or somewhat less. Ultimately, headings were preferred to shafts and adopted whenever practicable. They proved more convenient, as the men could work in front instead of at their feet; the men did not get wet from rain, and the ventilation was better. Headings were also less dangerous, as the tamping was less liable to be blown out. On the other hand, shafts were more easily tamped and required smaller charges of powder, the rock being already weakened by the excavation.

On an average, 4 tons of rock were blasted per lb. of powder, the extremes ranging from 5 to 2 tons. Generally the charges varied from 600 lbs. to 21,000 lbs. Experience determined the following coefficients for the formula given on p. 137:—

For ordinary shafts, coefficient $= \frac{1}{15}$ to $\frac{1}{20} = {\cdot}066$ to ${\cdot}05$;
For ordinary headings, coefficient $= \frac{1}{12} = {\cdot}083$.

In the exceptionally difficult case of a mine called a "rooter-out," the coefficient became ·1. This was a mine in which there was a natural joint on one side only, so that the rock had principally to be torn away from the solid mass. In such cases the lowest results were achieved, and, further, the stone displaced was usually in large masses requiring further breaking up, while in more favourable cases, the stone resulting from a blast was suitable for immediate use.

Figs. 105 and 106 are types of the best and worst kinds of mine respectively.

In tamping the headings, the powder was often built in for a few feet with a dry rubble wall, and the remainder of the galleries rammed with clay, obtained from a decomposed porphyry dyke in the quarry. The shafts were

[1] Hayter on Holyhead New Harbour, *Min. Proc. Inst. C.E.*, vol. xliv.

more readily tamped with quarry débris, stones, and clay thrown in and rammed. Occasionally, especially at the outset, a space was left round the charge, but it is believed that this was of little use.

The cost of quarrying the rock, including driving, powder, and sundries, was 4½d. per ton, and the cost of filling into waggons, including blasting the

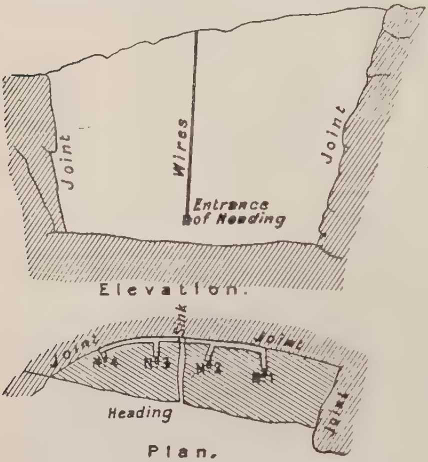


Fig. 105.—Mine at Holyhead.

Height of Face,	110	feet.
Length of Face,	140	,,
Length of Heading,	89	,,
Grip of Heading,	35	,,
Depth of Sink,	13½	,,

	Chambers.			
	No.1.	No.2.	No.3.	No.4.
Length,	12½′	12′	9′	2½′
Line of least resistance,	23½′	24′	25′	18′
Charge in lbs.,	4,500	4,000	3,000	1,500

Produce, upwards of 60,000 tons.

Depth of shaft, 44 feet.
Line of least resistance, 21 feet.
Charge, 900 lbs.
Produce, 2,000 tons.

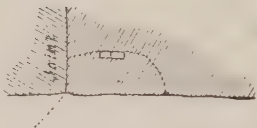


Fig. 106.—"Rooter-out" Mine at Holyhead.

large stones, was about the same. The prices, however, refer to the period of maximum output. The cost of driving the headings ranged from 10s. to 25s. per lineal foot, out of which the miners had to pay, on an average, about 2s. for powder, fuses, etc. The average length of heading driven was 5 feet per week with four men employed.

Concrete.

The subject of natural stone leads on almost insensibly to the kindred theme of artificial stone, for which an equally valuable, and a practically unlimited, field of usefulness exists. Those parts of maritime structures which are by far the most important and most prominent, are now constructed in concrete, in place of the elaborate masonry which characterised and distinguished the operations of past generations of engineers.

The cause and reason for this is not far to seek. Blocks of stone of large size are difficult to procure, expensive to dress, and equally expensive to convey and set in position. Smaller stones involve a multiplicity of joints. These, in themselves, are a source of weakness, but when their use is insepar-

[*To face p.* 144.

Fig. 105.—Fishguard Breakwater in Course of Construction.

ably combined with an intricate system of keyage and bonding, they prove doubly unsatisfactory and afford but an indifferent sense of security. What masonry, with its vertical and horizontal breaks and intersections, and its costly chisel-work, fails to ensure, is readily achieved by concrete, easily moulded while in a plastic condition to any required shape or outline, and deposited in position by the simplest means, with a minimum expenditure of time, money, and labour.

Concrete, as the term is generally understood amongst engineers, is an admixture of various mineral substances which, under chemical action, become incorporated into a solid body. Of its ingredients one group is inert, the other is active.

The inert group is called the *aggregate*, and it comprises any number of the following substances: slag, shingle, burnt clay or earthenware, broken stone, broken brick, gravel, and sand.

The active elements are hydraulic lime or cement and water. These constitute what is called the *matrix*.

The uses and applications of concrete are manifold. We have, however, to confine our attention to those points alone which are of pre-eminent importance and value in regard to maritime work, leaving other features and adaptations for treatment elsewhere.

The first thing to be noticed—it has already been animadverted upon in the earlier part of this chapter—is the necessity for an aggregate of high specific gravity. The reasons need not be repeated. From this point of view, heavy materials, such as slag and broken granite, are preferable to broken brick and sandstone.

Secondly, in order to ensure close adhesion, the aggregate should be rough and angular. Porous surfaces are admirable in this respect, but they do not generally appertain to heavy substances. However, the coarse crystalline texture of granite offers a sufficiently marked advantage over the smooth polished surfaces of flints and pebbles to constitute an excellent instance of what is meant by compliance with this requirement.

Thirdly, in order to reduce the number and volume of interstices, fragments of different sizes should be employed so that the smaller material may fill up the voids in the larger. At the same time, it is not desirable to use fragments of greater linear dimension than 4 inches, nor sand so fine as to be dust-like. It is usual to specify that the stone shall pass through a $1\frac{1}{2}$- or 2-inch ring, and that the sand shall be coarse and sharp. If fine sand be used, the grains cohere when watered and impede the introduction of the cement, besides requiring a greater quantity to effect the same complete envelopment.

Lastly, the aggregate should be perfectly clean and free from grease, clay, mud, and any other impurity whatever. Such substances have no adhesive value; they intervene between the parts which should come into contact, and are themselves readily soluble and removable by water, leaving the

mass in which they happen to be incorporated in a porous and laminated condition.[1]

The matrix is almost universally **Portland cement,** though hydraulic lime has been, and is still used, and also Roman cement. Hydraulic lime of a special character—the Teil lime—is a favourite with French engineers for sea work. It has been very largely employed in their works on the Mediterranean coast and along the English Channel, and, with one or two exceptions, seems to have answered satisfactorily. The use of Roman cement is limited to situations where rapidity of execution is essential, and where the hardening of the mortar is required to take place within a very short period. Neither hydraulic lime nor Roman cement has anything like the strength and durability of Portland cement.

Portland cement is an artificial product obtained by calcining, at a high temperature, an intimate compound of clay or shale with chalk or other limestone. In this condition, it contains a number of ingredients, of which the principal are lime, silica, alumina, and oxide of iron. These form about nineteen-twentieths of the whole, within the following limits, viz. :—

Lime, ...	60 to 64 per cent.
Silica, ...	20 to 24 ,,
Alumina, ...	6 to 10 ,,
Oxide of Iron, . . .	3 to 5 ,,

The remaining ingredients are magnesia, sulphuric acid, certain alkalies, and moisture. Of these, the magnesia should not be permitted to exceed 3 per cent., nor the sulphuric acid 1 per cent.

So great a variation has been manifested in the character of the different brands of cements emanating from the numerous manufacturers both in this country and abroad, and so much divergence of opinion has been exhibited in regard to standards and tests to be adopted for reference and comparison, that it has been felt desirable, and even necessary, to draw up a specification for general use among engineers. This has been done by the Engineering Standards Committee, and the result of their deliberations is embodied in the specification at the end of this chapter. It is not necessary, therefore, at this stage, to enter into the more general requirements of the model specification.

[1] Opinion is not, however, unanimous on this point. The following is a quotation from a paper on Reinforced Concrete, by Mr. M. Kahn :—" With reference to the sand used in concrete, some engineers specify that it shall be clean, sharp, and coarse. It has, however, often been demonstrated that sand need not be absolutely clean. Of course, it should not contain any foreign or vegetable matter, but a small percentage of dirt or loam will actually increase the strength of the construction, provided there is not more than 15 per cent. of loam in the sand. In other words, if a small percentage of loam is found in the sand it should not be washed out."—*Min. Proc. Liverpool Eng. Soc.*, vol. xxix.

Effect of Sea-water on Concrete.—The most vital consideration in regard to the matrix is the effect of sea-water upon concrete. On this point there is scope for much discussion and some ground for difference of view. On the one hand, there is abundant practical exemplification to demonstrate that Portland cement concrete is, in general, a thoroughly sound and durable material, in every way adaptable to maritime situations as elsewhere; on the other hand, there are indubitable instances of deterioration and failure. These instances obviously demand a searching inquiry, for in the absence of definite and authoritative refutation they must inevitably produce a feeling of doubt and uncertainty as to the propriety of using Portland cement in situations where its possible failure would entail consequences of the most serious nature.

The author has already, in another work, devoted some considerable space to a discussion of the subject,[1] and he does not feel that it will be considered incumbent upon him to restate the particulars and details of the investigation which led him to the conclusion that any defects which have exhibited themselves in the behaviour of Portland cement concrete in sea-water are due either solely and entirely to inherent deficiencies in the quality of the cement used, and in the materials with which it was mixed, or else, in part, to indifferent and imperfect manipulation.

Briefly stated, the facts in cases of known failure show that disintegration is (1) inaugurated by symptoms of expansion, and (2) subsequently accomplished by the solvent action of the sea.

The constituent elements of sea-water which have any effect upon cement are the sulphates and chlorides, especially the sulphate and chloride of magnesia. On coming in contact with lime, the magnesia in these salts is precipitated as a hydrate, with the simultaneous formation of chloride of calcium and sulphate of lime. Both these latter products are more or less soluble, especially the former, and the washing action of the sea soon occasions their removal.

But, in order that this interchange of constituents may take place, it is essential that there should be present a certain amount of lime, either actually in a free, uncombined condition, or, at least, in the form of a very unstable compound. The silicates and aluminates of lime produced during calcination, at the proper temperature for the manufacture of Portland cement, are all more or less fixed; at any rate there is abundant evidence to show that neither of these combinations necessarily breaks down in a maritime environment. It is usual, therefore, to attribute the change to some caustic lime which has failed to combine with the silica and alumina. The supposition is not out of consonance with observed phenomena. The slaking of lime causes expansion, with the formation of a hydrate which is readily soluble in water. One authority,[2] however, suggests that a truer explanation of the

[1] *Dock Engineering*, p. 123, *et seq.*

[2] Butler, *Portland Cement*, London, Spon, 1905.

expansion and disintegration is to be found in the formation of a "vitreous high-lime compound which slakes or hydrates so extremely slowly that it may be months before visible hydration even commences ; but in such cases hydration is accompanied by enormous expansion, the increase in bulk mounting to many times the original size of the mass, sufficient to cause the disruption and total disintegration of the previously set particles in the cement."

The precise nature of this "vitreous high-lime compound" is difficult to identify, but from whatever source the lime be forthcoming, there seems no doubt that it exercises a deleterious effect, and that measures should be taken to ensure its absence wherever possible, and, in the second place, to limit the action of sea-water to the outer skin or surface. This can only be achieved by making the concrete as impervious as possible, so that the bulk of it may be inaccessible to external influences.

Concrete made from sound Portland cement, mixed in proper proportions and thoroughly incorporated, is sufficiently impermeable for all practical purposes. It may not be absolutely water-tight—this is by no means essential [1] and can only be attained by the exercise of considerable trouble,—but it will display amply serviceable resistance to infiltration, which will thereby be rendered little more than superficial. Even in those cases where chemical changes have taken place, the evidence simply points to the deposition of magnesian salts in the outer pores of concrete from which the calcic hydrate has been removed. The magnesia is an inert substance, and while, in itself, an evidence of decomposition, its presence is attended by no additional ill effects ; in fact, it may even be claimed that it exerts a beneficial action in closing up pores which would otherwise remain open for the further penetration of sea-water into the interior of the mass.

In order to secure the highest degree of impermeability, a sufficiency of water must be used for mixing the concrete. An excess, of course, is objectionable, chiefly on the ground that it forms an incompressible volume in the fluid concrete, which passes away in evaporation, tending to leave the concrete porous. But, on the other hand, an insufficiency is attended by the evil that particles of cement may escape hydration, and this is a more vital consideration, in that there is a consequent lack of present cohesion and a source of future disturbance. It is better, on the whole, to water the concrete well rather than sparsely ; some proportion of moisture will be absorbed by the environment, the foundation, and adjacent work, and unless the mass be allowed to harden without undue abstraction of moisture, its strength will become impaired.

Speaking from long experience of a wide range of concrete work deposited in a tidal estuary, where the fluctuations of level are very great and where the circumstances are most propitious to the exercise of decomposing

[1] The remark, needless to say, applies to block work and not to reinforced concrete, the special treatment of which is described elsewhere (p. 121).

influences, the writer is convinced that the dangers attending the use of concrete work in maritime situations are often greatly and needlessly exaggerated. Ordinary care and discretion in the processes of mixing and deposition will prevent any evil consequences, provided, of course, the cement be of unassailable character, conforming in all respects to the requirements of the standard specification.

Another point affecting the use of Portland cement concrete in maritime work is the influence exerted by sea temperature upon its setting properties. The crystallisation or setting of cement is favoured by warmth and retarded by cold. The presence, therefore, of cold or warm currents in the sea exercises a corresponding effect upon the setting time, so that it is not a matter for surprise to find considerable variation at different places, and even at the same place at different seasons, in the period during which concrete work hardens. Of course, the more the time is prolonged the greater difficulty will be experienced in preserving the soft concrete from the chafing action of waves; but, on the other hand, it seems to be pretty clearly established that the slower the setting action, the greater the ultimate strength attained.

By way of completing this brief review of the subject, a model specification is appended, drawn up from a harbour engineer's point of view, and, therefore, containing several stipulations of a special character inapplicable to conditions elsewhere. The quality of the Portland cement, however, is strictly in accordance with the terms of the Engineering Standards Committee's specification, the clauses extracted from which are indicated by quotation marks.[1]

Specification.

The *aggregate* shall consist of gravel and broken stone of varying size mixed with sand, the quantity of sand being sufficient to fill completely the interstices in the larger material. The precise proportion of sand is to be ascertained by gauging the volume of water contained in a vessel which has been packed with the maximum amount of gravel and stone it can contain up to, and flush with, the level of the brim. No fragment shall measure more than 4 linear inches in any direction, and every piece must be capable of passing through a ring $2\frac{1}{2}$ inches in internal diameter. The length, breadth, and depth of the larger pieces must not be greatly unequal—*i.e.*, there must be an absence of long, flat, slaty slips, as also of smooth, water-worn pebbles. The stone must be heavy, weighing in the solid mass not less than 150 lbs. per cubic foot. Slag from ironworks may be used in place of, or in conjunction with, stone, provided it conform to the same conditions of weight and size and is not brittle or friable in any part. Both gravel and stone or slag must be

[1] The extracts are merely an abridgement from the standard specification, and do not pretend to embody the whole of the Engineering Standards Committee's requirements.

perfectly clean and free from admixture with any foreign substance, whether mineral or vegetable, and no gravel which has come as ships' ballast will be accepted.[1] The sand must also be clean; sharp, and not too fine—*i.e.*, it should all be retainable on a sieve of 32 S.W.G., having 900 meshes to the square inch. Dust and powder, as well as earthy and greasy matter generally, must be rigidly excluded.

Concrete described as x to 1 shall be understood to mean x parts by measure of gross aggregate as detailed above, combined with 1 part of Portland cement.

(Assuming that there is on an average some 35 per cent. of interstitial space [2] in the mixed stone and gravel, and allowing 5 per cent. margin to cover extreme cases, the quantity of sand required will be 40 per cent. The proportion of sand to cement should not exceed 3 to 1. Therefore, the minimum amount of cement will be 13 parts in 40.

Totalling the aggregates, we have :—

Gravel and stone,	60	parts
Sand, . .	40	,,
	100	,,
Cement, . .	13	,,
	113	,,

or a limiting ratio for the concrete of $8\frac{3}{4}$ to 1. For general work in bulk,[3] 8 to 1 is a serviceable proportion; for vertical facings, 6 to 1; for quay floors, 4 to 1; for quay steps and landings, 2 to 1.)

Displacers or *plums*—large stones and boulders of quality at least equal to that specified for the aggregate—may be inserted in the body of the concrete, forming mass or bulk work, provided that no two stones come within 6 inches of each other and that no part of any stone come within 6 inches of a moulded face. The rock or stone used for the purpose must either be brought fresh from the quarry, or, if old material from paving or building works, it must be thoroughly cleaned by picking and washing so as to free from all mortar, earth, and other accretions. The plums must be sound, hard, compact, and shapely, with no excessive elongation or attenuation and no cracks or flaws, and they must possess rough, preferably rugged, surfaces.[4]

The *matrix* shall consist of Portland cement manufactured by a firm of

[1] On account of the liability of a ship's hold to greasiness when used for mixed cargoes, especially in the case of oil in barrels, etc.

[2] Substantiated by experiment.

[3] 10 to 1 concrete is sometimes used for hearting purposes, but this proportion is somewhat extreme.

[4] No limits of size need be imposed.

good standing, and conforming in all respects to the tests and conditions stated below.

"The *cement* shall be manufactured by intimately mixing together calcareous and argillaceous materials, burning them at a clinkering temperature, and grinding the resultant clinker. No addition of any material is to be made after burning, other than calcium sulphate, or water, or both, and then only if not prohibited by the purchaser."[1]

As soon as possible after the delivery of the whole of any consignment on the works, *samples for testing* will be taken from twelve separate bags or parcels, in different positions. Equal portions of the several samples will be mixed together, and the cement so obtained will be considered as representative of the whole consignment and tested accordingly.

"Before any sample is submitted to tests for tensile strength, setting time, or soundness, it shall be spread out for a depth of 3 inches for twenty-four hours, in a temperature of 58 to 64 degrees Fahrenheit."[2]

Fineness.—"The cement shall comply with the following conditions of fineness. One hundred grammes (4 ozs. approx.) shall be continuously sifted for a period of fifteen minutes on each of the under-mentioned sieves, and in order of succession given below, with the following results:—

"(1) The residue on a sieve 180 by 180 = 32,400 meshes per square inch shall not exceed 14 per cent.

"(2) The residue on a sieve 76 by 76 = 5,776 meshes per square inch shall not exceed 1 per cent.

"The sieves shall be prepared from wire-cloth, and the diameter of the wire for the 32,400-mesh shall be ·0018 inch, and for the 5,776-mesh ·0044 inch. The wire cloth shall be woven (not twilled), the cloth being carefully mounted on the frames without distortion.

"The specific gravity of the cement, when presented for testing, shall not be less than 3·10."

The cement shall be delivered in packages marked with the manufacturer's name.[3]

Chemical Composition.—"The cement shall comply with the following conditions as to its chemical composition. The proportion of lime to silica and alumina when calculated (in chemical equivalents) by the formula $\frac{CaO}{SiO_2 + Al_2O_3}$ shall not be greater than 2·85 nor less than 2·0.[4] The percentage of insoluble residue shall not exceed 1·5 per cent.; that of magnesia shall not exceed 3 per cent.; and the total sulphur content, calculated as

[1] Some of the paragraphs of the standard specification are not transcribed in full.

[2] The limits of temperature throughout are applicable to a temperate climate. In other climates special arrangements are required.

[3] Any purchaser wishing to have the cement delivered in sealed bags, or in bags of any certain size, should so specify at the time of ordering.

[4] The molecular weight of lime = 56; silica = 60; alumina = 102.

sulphuric anhydride (SO_3) shall not exceed 2·75 per cent. The total loss on ignition shall not exceed 3 per cent.

Test for Tensile Strength (*Neat Cement*).—" The cement shall be mixed with such a proportion of water that the mixture shall be plastic when filled into the moulds used for forming the briquettes. The cement, gauged as above, shall be filled into moulds of the form required to produce briquettes of the shape shown in fig. 108, each mould resting upon a non-porous plate. In filling the moulds the operator's hands and the blade of the ordinary gauging trowel shall alone be used. The trowel shall weigh about 7½ ozs. No ramming or hammering in any form will be permitted. The moulds after being filled may be shaken to the extent necessary for expelling the air."

" Clean appliances shall be used for gauging, and the temperature of the water and that of the test-room, at the time the above operations are being performed, shall be from 58 to 64 degrees Fahrenheit."

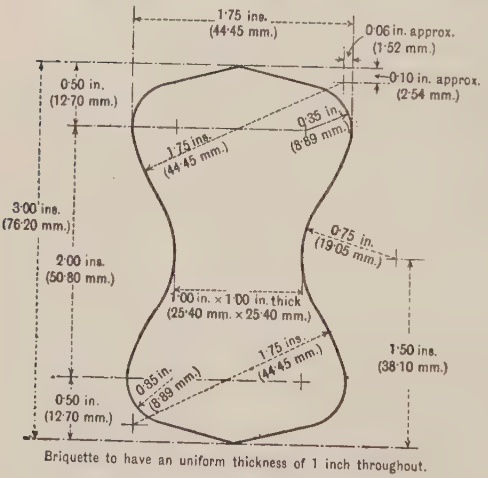


Fig. 108.—Standard Briquette.

" The briquettes shall be kept in a damp atmosphere for 24 hours after gauging, when they shall be removed from the moulds and immediately submerged in clean fresh water and left there until taken out for breaking After they have been so taken out, and until they are broken, the briquettes shall not be allowed to become dry. The water in which they are submerged shall be renewed every seven days, and shall be maintained at a temperature of between 58° and 64° F."

Briquettes of neat cement of the shape and dimensions shown in fig. 108, having a minimum section of 1 inch square, shall be tested for breaking at 7 and 28 days respectively, six briquettes for each period. The average tensile strength of the six briquettes shall be taken as the accepted tensile strength for each period. For breaking, the briquettes shall be held in strong, metal jaws of the shape shown in fig. 109. The load must then be steadily and uniformly applied, starting from zero and increasing at the rate of 100 lbs. in twelve seconds.

" The breaking strength of the briquettes at seven days after gauging shall be not less than 450 lbs. per square inch of section.

" The breaking strength of the briquettes at 28 days after gauging shall show an increase on the breaking strength at seven days, and shall be not less

than the number of pounds per square inch of section arrived at from the following formula:—

$$\text{“ Breaking strength at 7 days} + \frac{40{,}000 \text{ lbs.}}{\text{Breaking strength at 7 days}}.\text{”}$$

Test for Tensile Strength (*Cement and Sand*).—The cement shall also be tested by means of briquettes prepared from one part of cement to three parts by weight of dry standard sand, the briquettes being of the shape

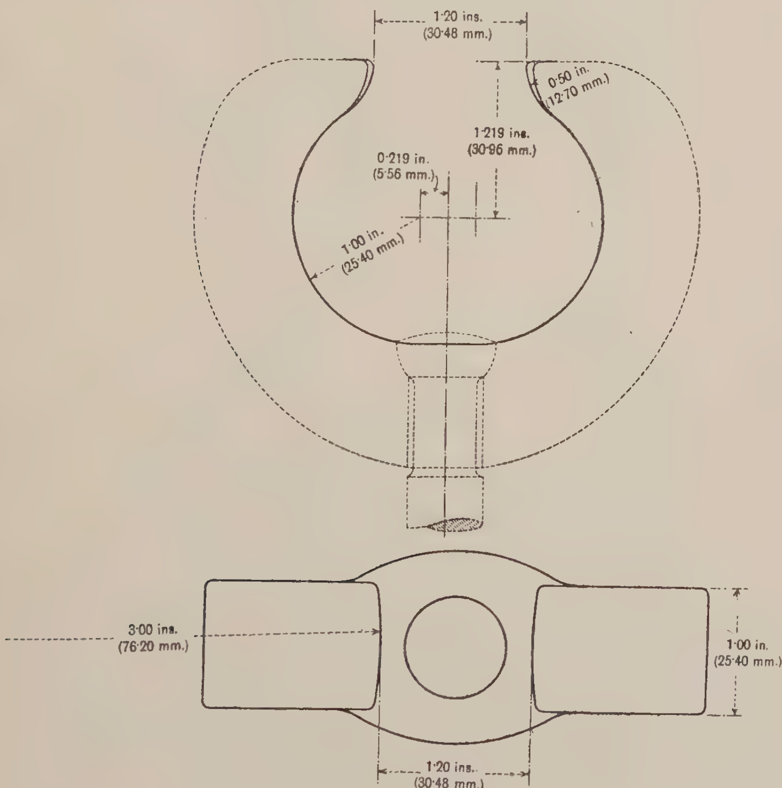


Fig. 109.—Standard Jaws for Briquette.

described for the neat cement tests. The proportion of water used shall be such that the mixture is thoroughly wetted, without excess.

“ After filling a mould, a small heap of the mixture shall be placed upon that in the mould and patted down with the standard spatula shown in fig. 110 until the mixture is level with the top of the mould. The last operation shall be repeated a second time and the mixture patted down until water appears on the surface; the flat only of the spatula is to be used, and no other instrument or apparatus is to be employed for this operation. The mould, after being filled, may be shaken to the extent necessary for expelling

the air. No ramming or hammering in any form will be permitted during the preparation of the briquettes, which shall be finished off in the moulds by smoothing the surface with the blade of a trowel."

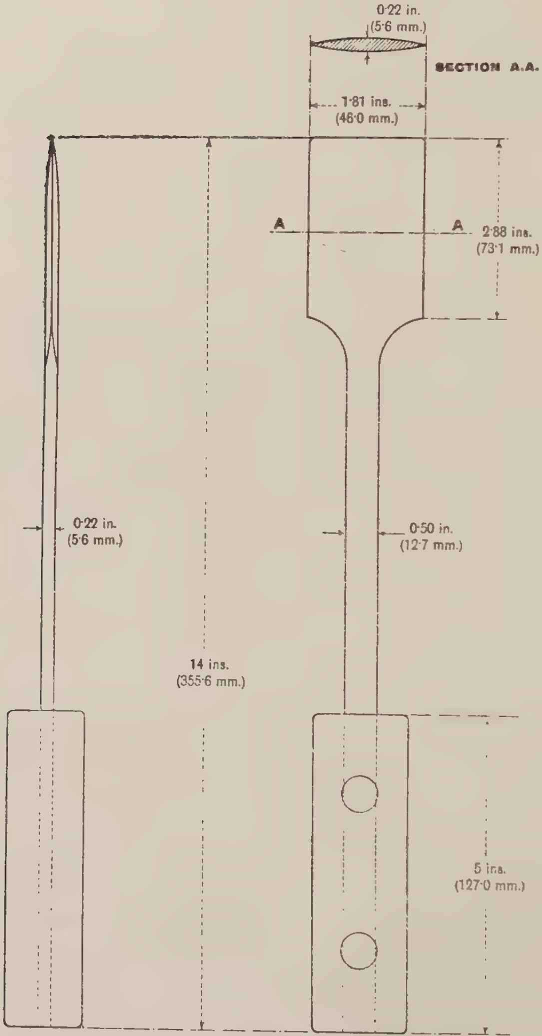


Fig. 110.—Standard Spatula.

The standard spatula shown above is of steel with a wooden handle securely riveted on. The total weight shall not exceed 12 ozs. (340 grammes) and the centre of gravity shall fall within 0·25 inch (6·4 mm.) of the centre of the length of the spatula.

The conditions of treatment until breaking and at the operation of breaking are to be as previously described for the neat cement briquettes.

"The breaking strength of the briquettes at seven days after gauging shall be not less than 200 lbs. per square inch of section."

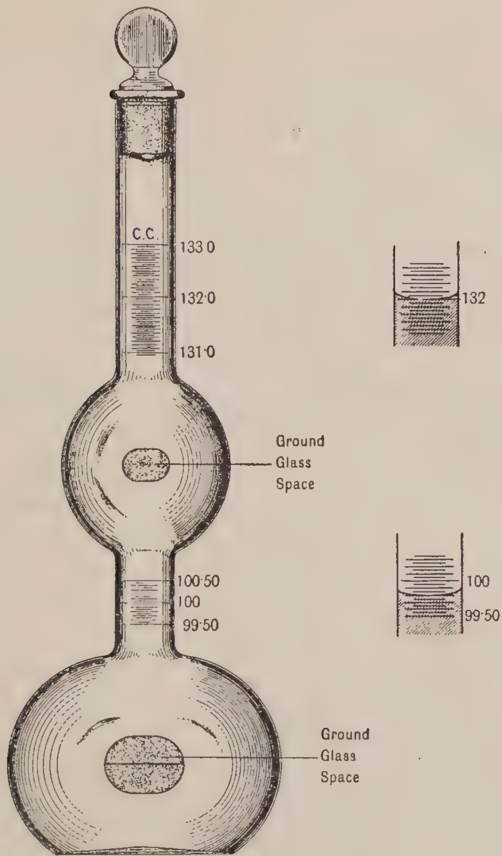


Fig. 111.—Specific Gravity Bottle.

Directions for Use.—Insert in a bottle, by means of a pipette without wetting the interior above the lower neck, 100 c.c. (approximately) of petroleum, paraffin, or other suitable liquid which has been freed from water. Place the bottle with the contained liquid in a tank of water of known temperature between 58° and 64° Fah. and allow it to remain there until the level of the liquid in the bottle remains constant, owing to its having attained the same temperature as the tank water. Read the level of the liquid in the bottle from the bottom of the meniscus, and make a note of the reading on the ground glass space provided for the purpose on the lower bulb. In the sketch at the side the reading is 99·8 c.c.

Now insert in the bottle, through a funnel, 100 grammes (accurately weighed) of the cement to be tested. Insert the stopper and tap the bottle carefully on a rubber, or other soft pad, to bring the bubbles of air to the surface. Replace the bottle in the tank, the water of which must be at the same temperature as in the case of the first reading, and when the level remains unchanged, again take a reading from the bottom of the meniscus. In the sketch at the side this reading is 132 c.c.

The specific gravity is then obtained as follows:—Deduct 99·8 c.c., the original volume of the liquid, from 132 c.c., which gives 32·2 c.c. as the volume occupied by 100 grammes of cement.

Then the specific gravity of the cement $= \frac{100}{32 \cdot 2} = 3 \cdot 105$.

" The breaking strength of the briquettes at 28 days after gauging shall show an increase on the breaking strength at seven days, and shall be not less than the number of pounds per square inch of section arrived at from the following formula :—

$$\text{" Breaking strength at 7 days} + \frac{10{,}000 \text{ lbs.}}{\text{Breaking strength at 7 days}}.\text{"}$$

" The *standard sand* shall be obtained from Leighton Buzzard, be thoroughly washed and dried, and shall pass through a sieve of 20 by 20 meshes per

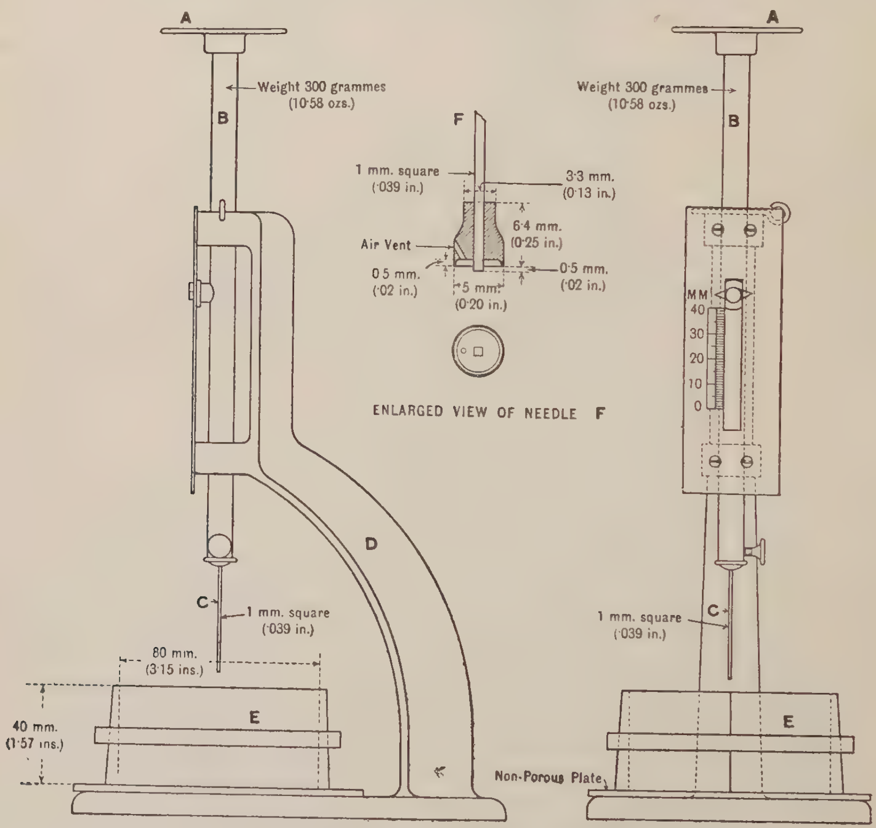


Fig. 112.—Needle for Cement Testing.

square inch, and be retained on a sieve of 30 by 30 meshes per square inch. The sieves shall be prepared from wire cloth, the wires being ·0164 inch and ·0108 inch in diameter respectively.

Setting Time.—" The cement shall be of one of three distinct gradations of *setting time*, which shall be designated as Quick, Medium, and Slow.[1]

[1] When a specially slow setting cement is required the minimum time of final setting shall be specified.

"Quick: Initial setting time not less than two minutes. Final setting time not less than ten minutes nor more than thirty minutes.

"Medium: Initial setting time not less than ten minutes. Final setting time not less than thirty minutes nor more than three hours.

"Slow: Initial setting time not less than thirty minutes. Final setting time not less than three hours nor more than seven hours.

Tests for Setting Time.—"The initial and final setting times of the cement shall be determined by means of the Vicat needle apparatus shown in fig. 112.

"For the determination of the initial setting time the test block, confined in the mould and resting on the plate, shall be placed under the rod bearing the needle; when the latter shall then be lowered gently into contact with the surface of the test block and quickly released, and allowed to sink into the same. This process shall be repeated until the needle, when brought into contact with the test block and released as above described, does not pierce it completely. The period elapsing between the time when the cement is filled into the mould and the time at which the needle ceases to pierce the test block completely shall be the initial setting time.

"For the determination of the final setting time the needle C of the Vicat apparatus shall be replaced by the needle F, shown separately. The cement shall be considered as finally set when, upon applying the needle gently to the surface of the test block, the needle makes an impression thereon, while the attachment shown in fig. 112 fails to do so."

The test blocks shall be mixed as previously described.

Test for Soundness.—The cement shall be tested for soundness by the Le Chatelier method, the apparatus for which is shown in fig. 113.

"In conducting the test the mould shall be placed upon a small piece of glass and filled with cement gauged in the manner and under the conditions referred to, care being taken to keep the edges of the mould gently together whilst the operation is being performed. The mould shall then be covered with another glass plate, upon which a small weight shall be placed, and the whole shall then be immediately submerged in water at a temperature of 58° to 64° F., and left there for 24 hours.

"The distance separating the indicator points shall then be measured and the mould again submerged in water at 58° to 64° F., which shall be brought to boiling point in 25 to 30 minutes and kept boiling for six hours. The mould shall then be removed from the water and allowed to cool and the distance between the points again measured; the difference between the two measurements represents the expansion of the cement. When the sample has been aerated for 24 hours in the manner described, the expansion as above determined shall not exceed 10 millimetres. In the event of the cement failing to comply with this test, a further test shall be made from another portion of the same sample after it shall have been aerated for a total period of seven days in the manner before described, when the expansion determined as above shall not exceed 5 mm."

The foregoing tests shall, as far as possible, be made within fourteen days from full delivery of each consignment of cement, and any consignment, the samples of which do not prove satisfactory in testing, shall be rejected.

Some of the test briquettes made as described above will be kept intact for a period of six weeks, and these will be examined from time to time. Should any of them show signs of cracking or disintegration within six weeks from date of gauging, the whole of the cement represented by the defective briquettes will be condemned.

The whole of any individual consignment of cement must be delivered on the site of the works sufficiently in advance of its intended use to allow of the foregoing tests being completed and adjudicated upon.

Contractors, or others using the cement, must provide on the site a suitable water-tight, dry, wooden-floored store, capable of accommodating one-hundred ton lots so as to be kept distinct from each other. The cement

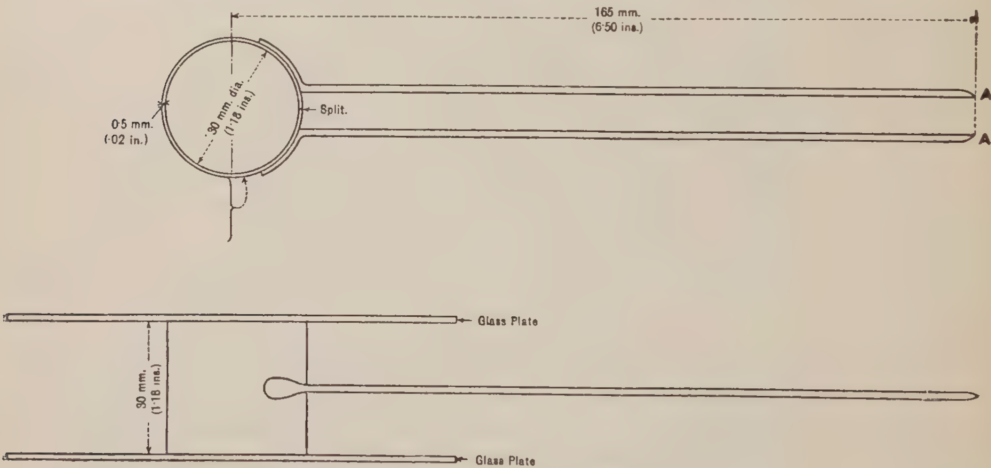


Fig. 113.—Apparatus for Le Chatelier Test.

The apparatus for conducting the Le Chatelier test (fig. 113) consists of a small split cylinder of spring brass or other suitable metal of 0·5 millimetre (·02 inch) in thickness, forming a mould 30 millimetres (1·18 inches) internal diameter and 30 millimetres high. On either side of the split are attached two indicators with pointed ends A A, the distance from these ends to the centre of the cylinder being 165 millimetres (6½ inches).

is to be spread out over the floor to a depth not exceeding 3 feet, for a period of three weeks, immediately prior to using, and is to be turned over twice a week.[1] In humid weather the cement is to be protected from an excessive absorption of moisture. The stock must never be allowed to fall below six weeks' requirements.

Cement which has failed to come up to the requirements of the specifica-

[1] In view of modern methods of treating the cement prior to delivery, this clause is not now generally necessary.

tion must be removed forthwith, and no cement must be used until a certificate of its efficiency has been issued.

For the purpose of *gauging* the ingredients for concrete, strongly constructed measuring boxes are to be provided.

The mixing of the concrete may be done either by hand or by a machine of approved make.

For hand-mixing the following procedure is to be adopted. A wooden platform must first be laid down, and upon this a cubic yard of the aggregate will be deposited and spread out into a uniform layer 12 inches in depth. This layer is to be evenly covered by the proper proportion of cement, and the whole turned over three times dry. Then water is to be added, applied through a rose-ended sprinkler, while the concrete is again turned over three times wet. On no account is the concrete to be deposited until perfect incorporation has been effected.

The quantity of water used must be adequate to bring the concrete to a viscous condition, of the consistency of slime. There must not be any excess, however, such as would wash the cement from the aggregate.[1]

In windy weather suitable screens are to be provided to prevent loss of cement.

In frosty weather no concrete is to be prepared without definite sanction and under appropriate conditions.[2]

No concrete is to be allowed to stand after being mixed, but must be used forthwith. It may not be thrown into foundations from a greater height than 6 feet, and it must be deposited in such a manner as to secure homogeneity and compactness.

Concrete work shall, as far as practicable, be carried on continuously in a series of layers not exceeding 3 feet in thickness, extending over the whole of the site. Where this is impossible, the higher part of it must be racked or stepped downwards to meet the lower, in steps about 5 feet high by 2 feet broad, and vertical joints must be avoided unless necessitated by particular requirements.

The scum arising from the concrete is to be allowed to drain away, and any that settles on the surface of a layer is to be carefully removed. After a layer has set, for which purpose two days must be allowed, an additional layer may be deposited, but not before the surface of the former layer has been well picked, washed with clean water, and brushed.

The surfaces of all brickwork, masonry, or concrete, on or against which concrete is to be laid, must be thoroughly cleaned and wetted immediately before the concrete is applied.

All wooden moulding boards are to have their surfaces paid over with

[1] This applies to concrete used in situations free from water. For deposition under water, special measures are required which cannot be covered by a general specification.

[2] Such as the use of very salt water and, possibly, of sugar. The work will also require covering.

oil, or a suitable composition, to prevent the concrete from adhering to them.

The exposed surfaces of all concrete work must present a fair and smooth appearance where such is desired, and any superficial irregularities must be made good with mortar composed of 1 part Portland cement to 2 parts of clean, sharp sand.

Where a facing of higher quality concrete is to be worked on to a lower quality backing, the division between the two portions is to be formed by a movable hand shutter. The two qualities (as 8 to 1 and 6 to 1) are to be deposited simultaneously and the shutter gradually raised, so that there may be thorough incorporation and the absence of any break or joint.

Inspection.—Finally, it may be observed that owing to the dependence of sound concrete upon perfect manipulation, both in mixing and in depositing, too much stress cannot be laid upon the desirability of appointing a trustworthy and competent man to personally supervise all concreting operations. In the case of work done by contract, it is a most essential step; in this way alone can the character of the workmanship be guaranteed, and without that, the best materials may prove practically worthless.

As a matter of interesting comparison, the conditions laid down in a modern Japanese specification are appended.

Concrete in Blocks at Osaka Harbour Works, Japan.[1]

The proportions of the concrete were as follows:—

Portland cement, .	25 lbs. to 1 cubic foot of sand.
Sand, . . .	2 } by volume.
Gravel, . . .	3 } by volume.

Since each block contained 120 cubic feet, the corresponding ingredients were:—

Cement, ...	1,500 lbs.
Sand, ...	60 cubic feet.
Gravel, ...	90 cubic feet.

Samples of the cement were taken from 2 per cent. to 5 per cent. of the barrels of every cargo. Extracts from the principal clauses of the specification are as follows:—

Chemical Analysis.—"If a sample of cement shows by chemical analysis that it contains either more than 1 per cent. of anhydrous sulphuric acid, or a trace of calcium sulphide, or more than 3 per cent. of magnesia, or more than 4 per cent. of ferric oxide, or that the hydraulic index is less than 42, the cement shall be rejected."

Setting.—"Cements which begin to set in less than one hour or finish

[1] Shima on Osaka Harbour Works, *Trans. Am. Soc. C.E.*, vol. liv.; Int. Eng. Conf., 1904.

setting in less than three hours or later than twelve hours, shall be rejected."

Tensile Strength.—"The tensile strength of the neat cement briquettes after seven days shall be not less than 285 lbs. per square inch, and after twenty-eight days not less than 500 lbs. per square inch; that of the standard sand mortar briquettes after seven days shall be not less than 110 lbs. per square inch, and after twenty-eight days not less than 215 lbs. per square inch. (The standard sand mortar consists of 1 part of cement to 3 parts of standard sand.)"

The sand was obtained at the mouth of the River Yamato; its grains were clean, sharp, and angular. The sand was screened before being used, on a sieve of $\frac{3}{16}$-inch square mesh.

No special variety of gravel was specified, but it was limited to that from sea beaches. It was obtained mostly from the north-western coast of Osaka Bay. The particles were hard and clean, but not very sharp. They were screened between 2-inch and $\frac{3}{8}$-inch sieves.

CHAPTER VII.

BREAKWATER DESIGN.

Importance of Breakwaters—Regimen—The Sea Wave—Form, Height, and Length—Breaking Waves—Dynamical Value—Measurement of Wave Stroke—Dynamometers—Recorded Pressures—Instances of Wave Action—Classification of Breakwaters—Comparison in Cost of Construction and Maintenance and in Efficiency—Conditions of Stability—Stresses in Wall Breakwaters—Summation of Type Characteristics—Examples of Breakwater Design at Portland, Plymouth, Holyhead, Genoa, Marseilles, Algiers, Sandy Bay, Tynemouth, and elsewhere.

The most important work, as also the most prominent and fundamental feature, in connection with artificially sheltered harbours and roadsteads, is the **Breakwater.** As the name implies, its function is to break up and disperse heavy seas, preventing them from exerting their destructive influence upon the area inclosed for the reception of shipping. Manifestly, then, a breakwater must be characterised by great strength and stability. The safety of helpless vessels and the efficiency of the harbour as a place of refuge are bound up in the essential permanence and immobility of the breakwater.

Before proceeding to an investigation of the principles which underlie the design of breakwaters and by which these objects may be attained, we have to pass in review the conditions and environment to which such structures must conform and the general circumstances attending their construction and maintenance.

Regimen of Breakwaters.—Structures erected within the domain of the sea and submerged for the greater part of their bulk, if not altogether, are subjected to physical experiences of a nature very different from those which are characteristic of structures on land. The fact of immersion materially modifies the effect of gravity upon a body, reducing its apparent weight to a very considerable extent. That this condition must be applicable to maritime structures is obvious, unless, indeed, the foundation be absolutely impervious and there be an entire absence of ducts for the penetration of water—conditions which, in many cases, are quite unrealisable, and in most are so imperfectly guaranteed as to render them inacceptable as working hypotheses. The solvent properties of water combined with the extreme mobility of its particles, cause it to act in a most prejudicial and injurious manner upon much of the material used in breakwaters, as well as upon the foundation itself, and these merely mechanical effects are supplemented and aggravated by physical and molecular changes, resulting in deterioration in strength and durability. The intensity of the external forces which make

for disruption is enormous, exceeding beyond all comparison the power of the wind on land structures. Wave agency is a thousandfold more potent than the most intense atmospheric movement. There are, moreover, insidious denizens of the sea, infesting it by millions, which, by their concerted action, are capable of undermining the hardest and soundest building materials, and that in the most secret and surreptitious manner, the damage being as unsuspected as it is irremediable.

Such inimical natural phenomena constitute the normal and characteristic environment of all maritime structures. They are bonded together, as it were, in an offensive alliance to urge incessant and unrelenting war upon man's handiwork—sapping, wearing, battering, making subtle inroads and open breaches, working now by patient effort, long sustained, and now by sudden, prodigious feats, month after month, year in and year out, knowing neither truce nor armistice.

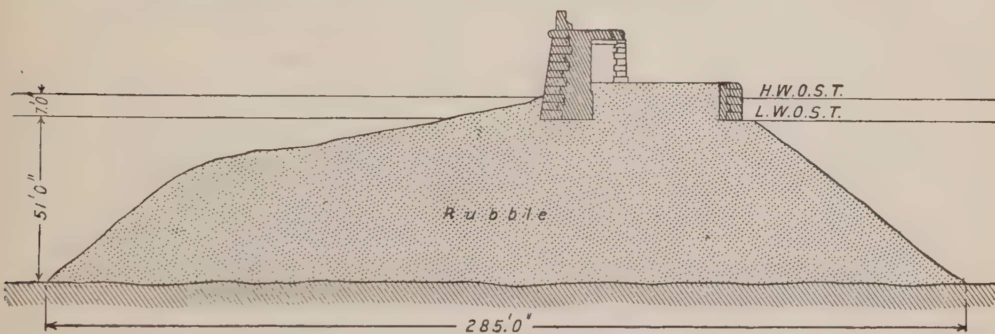


Fig. 114.—Section of Portland Breakwater.

The Sea Wave.—But by far the mightiest of the forces arrayed against the harbour barrier is the sea wave. This mysterious product of wind and water is endowed with tremendous disruptive power. It acts with all the magnipotent impulse of a huge battering ram, while, at the same time, it is equipped with the point of the pick and the edge of the wedge. It is, in fact, one of the most complex, the most volatile, the most pertinacious, and the most incomprehensible of natural forces.

From an engineering point of view, we have little to do with abstract theories of wave formation. Mathematically, the subject is too abstruse for any but very accomplished and capable mathematicians, and the intricacies of calculation are interesting only as academical exercises. Many of the theories advanced are merely tentative and lack substantial corroboration; others, while generally accepted, are still the subjects of speculation and inquiry. Thus, no useful purpose would be served by pursuing an investigation into the laws and phenomena of water undulation. Students who wish to do so, however, may consult the articles on Wave and Tide in the *Encyclopedia Britannica* or the *Encyclopedia Metropolitana*. The late Sir George Airy, the distinguished Astronomer-Royal, also wrote a treatise on

Tides and Waves, and this, with the works of Scott Russell and Weber, and later, of Gaillard and Wheeler, afford sufficient scope for reference.

Yet, while disclaiming any intention of probing into the depths of abstruse speculation, we cannot abstain from alluding in general terms to those principles of wave action which have reference to their physical effects upon engineering structures. Such information is essential to an appreciation of the problems of breakwater design.

For the present purpose, it suffices to state that water waves have conveniently been divided into two classes—viz., waves of oscillation, without forward motion, and waves of translation, possessing it. Yet, in spite of this distinction—a purely artificial one—it seems probable that all waves are more or less waves of translation, causing the particles of which they are composed to advance permanently to some slight extent, at least. So far as sea waves are concerned, those which possess the power of exerting any appreciable effect on the stability of maritime works are undoubtedly waves of the second division.

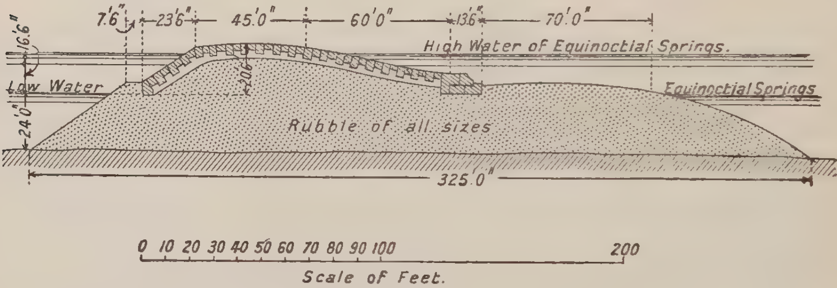


Fig. 115.—Section of Plymouth Breakwater.

Form of Waves.—The formation of storm waves takes place in the open sea, and their inception is, of course, due to the wind. The outline assumed is extremely variable, depending both upon the length of the undulation and its period, the last-named being the interval of time in which the wave traverses a distance equal to its length. The crest or summit of a wave is sometimes rounded, sometimes acute, and, in either case, it attains a height above mean sea level greater than the depth of the trough below it. In a swell in the open sea, the profile of a wave perhaps most nearly resembles a sinoidal curve, the slope directly exposed to wind action being more gradual and less steep than the leeward slope.

Under the conditions of modern investigation, however, as exemplified in the researches of Weber, Scott Russell, Enry, and Aimé, the hypothesis has been advanced that there is an orbital movement in waves, each particle of which they are composed pursuing a regular geometrical path. The precise nature of the path depends upon local conditions. When the depth of water is sufficiently great—that is, where the depth is at least equal to the length of the wave from crest to crest—the motion of the particles of water is rotary

along the circumference of a circle, as shown in fig. 116. The wave is one of oscillation, and each particle completes a revolution, returning approximately to its initial position. The profile of the wave, then, is a cycloidal curve traced

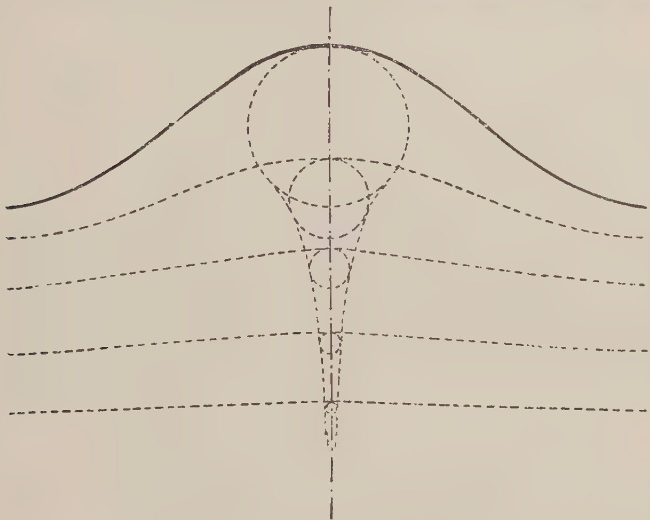


Fig. 116.—Wave in Deep Water.

out by a generating circle, which constitutes the orbit of the surface particles. Accordingly, the actual momentary direction of motion of each of the particles is independent and variable. Thus, at the crest, the motion is

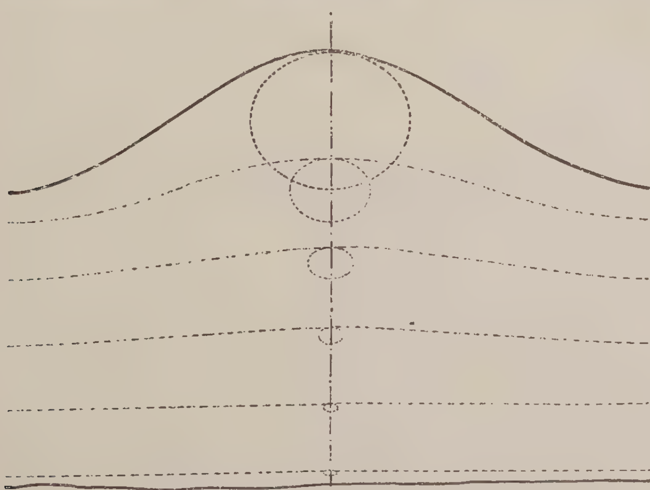


Fig. 117.—Wave in Shallow Water.

horizontally forward ; in the trough, it is horizontally backward ; whilst at the midpoint between these extremes, it is purely vertical. Below the surface level the circular paths diminish rapidly to an insensible minuteness. At a

depth equal to the length of the wave, the displacement of the water particles is $\frac{1}{535}$ of that of the surface particles, and at double the depth the ratio is reduced to $\frac{1}{286,690}$.

In shallow water of uniform depth—that is, in water the depth of which is less than the length of the wave—the orbit of the water particles is approximately elliptical with the major axis horizontal, as shown in fig. 117. The centre of the orbit lies slightly above the position of rest. With this exception, the same dispositions hold good as in the previous case as regards the movement of the particles. The ellipses of movement become flatter as the distance below the surface increases, until finally at the bottom there is horizontal motion only. In water which has a depth of only one-tenth of the length of the wave, the ratio of the elliptical axes at the surface is about $\frac{7}{12}$, and at nine-tenths of the depth it is $\frac{1}{16}$.

When, instead of remaining uniform in depth, the water in which a wave is travelling becomes increasingly shallow (fig. 118), the orbits of the particles

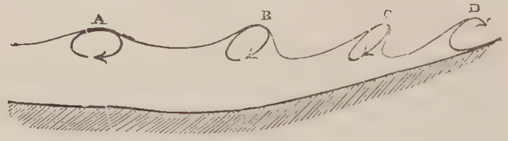


Fig. 118.

of water become correspondingly distorted. Owing to the friction exerted along the bottom, the major axis of revolution acquires an inclination to the horizontal, which is continually augmented. The wave ceases to be purely oscillatory: it undergoes a gradual transformation. The front of it becomes steeper than the back, the crest gaining more and more upon the trough until it actually overhangs. Then it falls forward and breaks into surf. At this point the wave is altogether a wave of translation, and the forward motion of the particles is exactly equal to the velocity of the wave. It is in this phase that waves possess their most formidable potency.

Any sudden change in the level of the ground over which a wave is travelling is capable of producing the disruption of the wave. This effect is not confined to shallow reaches, but extends to depths as great as 16 to 20 fathoms, or even more, in the open sea. Thus, on the Herreca reef, seven miles from land, breakers are apparent in tempestuous weather in a depth of 90 feet of water.

The "Air" Breakwater.—A strikingly novel proposal for annulling the disruptive force of waves has recently been put forward by Mr. Brasher, an American engineer. His proposition is that the wave may be dispersed in the undulatory stage by the introduction of a continuous stream of air from a perforated pipe lying along the sea bottom, the air being conducted under pressure to a distance from the shore sufficient to enable the discharge to anticipate the generation of the translatory stage of the wave.

A trial installation was made in connection with a pier at El Segundo, California, consisting of two 3-inch air supply mains running seaward a distance of 145 feet with a 4-inch discharge pipe, connecting their extremities, which were 120 feet apart. The following statement as to the result of the experiment is extracted from an article in the *Engineer* :—

"About the middle of January last, the Pacific Coast was swept by a violent storm that did an enormous amount of damage to property along the coast. The installation at El Segundo had, fortunately, been made ready for service some weeks earlier, and was available for trial at the time of the gale. The waves ranged in the neighbourhood of from 12 to 15 feet high, and followed very quickly upon one another, and it is considered certain that the seas would have seriously injured the pier, if they had not carried it away, had it not been for the intervention of the air breakwater. For 23 hours the compressed air was fed steadily into the perforated conduits under water, and the action of the rising bubbles robbed the waves of their harmful surge, and left them so subdued that they rolled shoreward without, it is said, even jarring the wharf when the storm was at its worst. The total outlay for the running of the compressors was about £12."

The installation was of too slight a character and its basis too indefinite to allow of any decided opinion as to the merits and efficacy of the treatment. There may be some beneficial interference with the normal propagation of the wave, but it remains to be demonstrated more convincingly how and why the generative impulse can be overcome in the case of heavy rollers by so slight a means as the injection of air bubbles.

Height of Waves.—The inception of waves being due to the wind, their development manifestly depends upon the extent of surface acted upon Waves generated without restriction are capable, under propitious circumstances, of attaining a very high degree of development, both as regards height and length. On the Lake of Geneva, for instance, storm waves are stated to reach a height of 10 feet ; in the North Sea, from 12 to 15 feet ; in the Mediterranean Sea, from 15 to 20 feet ; in the Bay of Biscay, from 25 to 30 feet ; in the open Atlantic, from 30 to 40 feet [2] ; and in the Pacific (off Cape Horn and the Cape of Good Hope), from 50 to 60 feet. Other estimates of a much higher nature have been made, but it is open to question whether they have not been influenced by an unconscious tendency to exaggeration on the part of the observer, due to the inspiring nature of the spectacle, or been founded upon mistaken and erroneous data. There is, in such cases, a strong and an acknowledged inducement to use picturesque language and to speak

[1] "The Brasher Air Breakwater," *The Engineer*, May 19, 1916.

[2] Dr. Vaughan Cornish, in his book on "Waves of the Sea," says of his own observations and those of others that they "indicate that anywhere in the North Atlantic with sea room of from 600 up to certainly 1,000 and, perhaps, 2,000 miles, the height of the large waves during ordinary strong gales is practically constant, being not less than 43 feet." Page 60.

of waves as "mountains high," to which, of course, numerical values are given to correspond as far as possible. Indeed, viewed at close quarters, a formidable wall of water towering suddenly above the spectator's line of sight, even when on the upper deck of a vessel, can hardly fail to produce an illusory sense of enormous magnitude and overwhelming menace. So far, however, as unquestionable records go, it may safely be asserted that 50 feet is about the maximum height attainable by unbroken waves, and this view is supported by the opinions and testimony of Sir George Airy, Captain Scoresby, and other observers.[1]

The heights of waves breaking against the cliffs and headlands of a rocky coast do not, of course, come within this category. The summits of columns of water thrown up by the force of impact attain, as might be expected, to

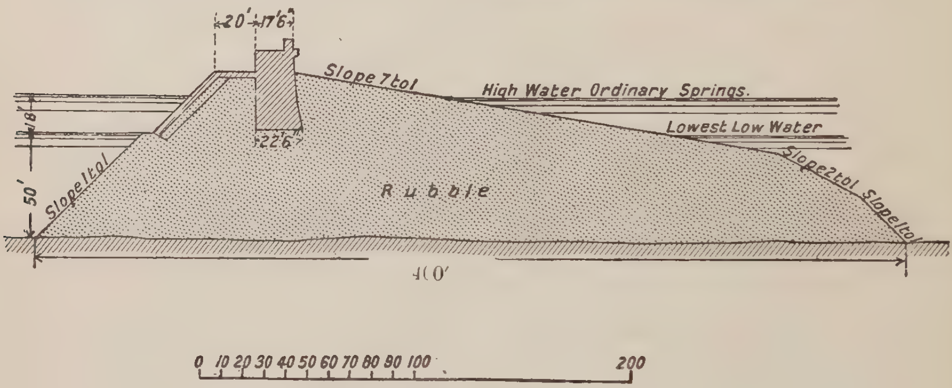


Fig. 119—Section of Holyhead Breakwater.

much greater altitudes. The effect is particularly noticeable in the case of lighthouses and prominences with vertical or nearly vertical faces. Thus, at The Hague heights of 75 feet, at Bell Rock 100 feet, and at Eddystone

[1] In this connection, the following extract from *Knowledge* of October, 1912, will be of interest :—

"*Ocean Waves.*—In the Monthly Meteorological Chart of the Indian Ocean and Red Sea for August, some very interesting particulars are given of high ocean waves. It appears that the abnormally high solitary sea is the most dangerous, and that these are not confined to tropical seas, but are occasionally met with near our own coasts. Thus, in 1897, at the entrance to the English Channel, the steamer 'Millfield' encountered a sea which washed overboard the upper bridge, the funnel, and the boats, and had her fires extinguished by 8 feet of water, which entered by the openings on deck. Another case mentioned is that of the 'Brandenburg,' which shipped a tremendously high wave, estimated at 65 feet in height, which stove in the crow's nest, constructed of quarter inch steel plating, at 50 feet above sea level."

It is desirable to make the comment that the height at which damage is done to a ship's structure is not necessarily a reliable criterion of the actual height of the wave, as when this encounters any obstacle it is deflected upward to a height much greater than that which would be attained in the free state. See next paragraph above.

150 feet, have frequently been recorded, and Lord Dunraven has observed heights of 150 feet on the precipitous South-West Coast of Ireland.

On the basis of the evident connection existing between the development of waves and the generating distance, Stevenson, the eminent harbour engineer, devised an empirical formula to determine the height of waves from the *Fetch*, or extent of sea available for the purpose of generation. Taking H as the height, in feet, of the wave, and F as the length, in miles, of the fetch, he found that approximately

$$H = 1{\cdot}5\sqrt{F}, \qquad (\alpha)$$

or, more closely, for short fetches of less than 30 miles,

$$H = 1{\cdot}5\sqrt{F} + (2{\cdot}5 - \sqrt[4]{F}). \qquad (\beta)$$

The following instances afford a comparison of the results of calculation based upon these formulæ, with the values obtained from actual observation.

HEIGHT OF WAVES.

Place of Observation.	Length of Fetch. Nautical Miles.	Observed Height of Waves in Feet.	Calculated Height in Feet.	
			Formula (α).	Formula (β).
Scalpa Flow, . . .	1·0	4·0	1·5	3·0
Firth of Forth, . .	1·3	1·8	1·8	3·2
Lough Foyle, . . .	7·5	4·0	4·1	4·96
Clyde, ...	9·0	4·0	4·5	5·25
Colonsay, . . .	9·0	5·0	4·5	5·25
Lough Foyle, . . .	11·0	5·0	5·0	5·7
Anstruther, . . .	24·0	6·5	7·5	7·7
Lake of Geneva, . .	30·0	8·2	8·2	8·37
Buckie, ...	40·0	8·0	9·5	..
Douglas, I.O.M., . .	65·0	10·1	12·0	..
Kingstown, . . .	114·0	15·0	16·0	..
Sunderland, . . .	165·0	15·0	19·3	..
Peterhead, . . .	400·0	22·6	30·0	..

It is manifest, however, that waves cannot attain their full development where there is inadequate depth. No wave can have a height greater than the depth of water through which it passes. Consequently, the intervention of shoals in the path of a wave serves to limit its size. Reefs and sandbanks, even though entirely submerged, materially reduce the range of undulation, and the length of fetch must be gauged accordingly. Thus the effective length of open sea may be much less than the apparent length.

But, even with this restriction, the fetch is far from being an exact or reliable indication of wave height in every locality. It is true that the maximum height attainable can be calculated therefrom with some approach to accuracy; but the fact must not be overlooked that winds may not always, or often, or indeed ever, blow along the line of maximum fetch. There is a stretch of 500 miles of open sea leading to the harbour of Kurrachee with

unrestricted depth, yet the highest waves—those from the south-west—are said not to exceed 15 feet. Other instances might be adduced—such as Peterhead in the preceding table—to show that the maximum fetch alone is by no means an infallible criterion of wave height. There is also another point of no slight importance. Not only is it possible for the severest gales to blow from some other quarter of the compass than that lying in the direction of the greatest fetch, but it is also a matter of experience that heavy rollers are frequently deflected so as to reach a point on the coast which does not lie upon their direct path. This is exemplified in the case of a headland and bay in fig. 120, and the same effect is noticeable at the pierheads of artificially

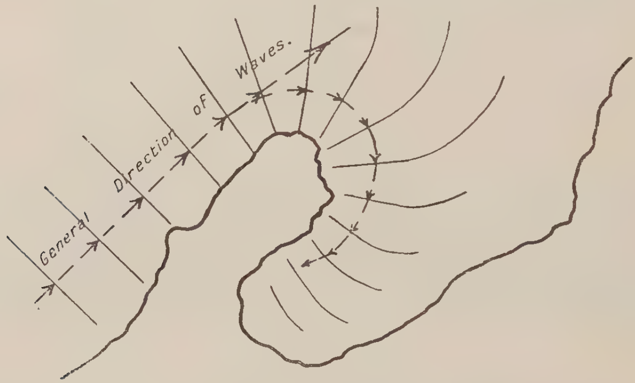


Fig. 120.—Deflection of Waves by Headland.

sheltered harbours. Islands also act as pivots in many cases, causing the waves to wheel round and break upon their leeward shores. The result is due, no doubt, to the retardation produced by shallowing ground upon the nearer portion of waves approaching a coast obliquely, or running parallel thereto.

Furthermore, the convergency due to narrowing inlets tends to accentuate to a greater degree even, the eccentricities of wave development. Pent between lateral arms drawing gradually closer together, the volume of water is raised above the level it would normally assume, and so gives rise to breakers of a character equally marked.

As a corollary to what has been said, it is evident that waves of great height cannot reach any coast-line, and for that matter, any artificial barrier, unless there be an unbroken extent of deep water penetrating close into them.

Length of Waves—The length of waves is a feature which seems to be independent of the height, though it is connected in some way with the amount of exposure to wind action, and it influences the force of the wave. In the Atlantic Ocean waves of from 500 to 600 feet between crests have been observed, while in the Pacific they are stated to reach anything from 600 to 1,000 feet.

The length of waves, however, in the open sea is a difficult matter to

determine satisfactorily, owing to the absence of any reliable linear standard. Alongside jetties and piers, the obstacles in the way of exact measurement are not so great, and serviceable computations may be made with the aid of Bertin's formula. Observing the length of time in seconds which elapses between the passage of the same point by two successive crests—in other words, the period of the wave, calling this period P and the length of the wave in feet L,—we have

$$L = \frac{P^2 g}{2\pi};$$

or, fairly approximately,

$$L = 5\tfrac{1}{8} P^2.$$

The length of the wave in conjunction with the depth of water determines the speed of movement of the wave and, conjointly, the velocity of the particles of which it is composed. The relationship existing between these elements will be discussed a little later.

Breaking Waves.—We have now to consider the manner in which a wave acts upon any fixed obstacle in its path, whether it be the beach upon which it is spent or an artificial barrier which causes its abrupt collapse.

Dealing first with the oscillatory wave, and assuming that it reaches a wall or other obstruction having an abrupt vertical face, we find that it is reflected in the manner indicated by fig. 121. The particles of water in contact with the wall (A) move up and down through a height which is twice the height of the original wave, as also do the particles in the trough (C) half a wave-length distant. At a point (B) midway between the trough and the wall—that is, one quarter of a wave-length from either—the particles move horizontally backwards and forwards, while at intermediate points the path of the particles is inclined at various angles. The whole motion, in fact, is the inverse of that which occurs in the unobstructed wave.

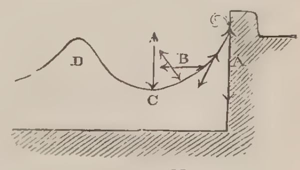


Fig. 121.

When, on the other hand, without meeting with any abrupt obstacle, the wave advances into rapidly shoaling water, its energy is communicated to smaller and successively decreasing masses. Consequently there is a tendency to produce in those masses an agitation of increasing violence. But this effect is generally diminished, and sometimes entirely counteracted, by the loss of energy due to friction along the bottom, and to surging. On the other hand, the influence of concentration arising from funnel-shaped inlets is clearly to intensify the agitation, and the same effect is producible by submerged rocks with deep narrow gorges between, in passing through which the water is heaped up into masses of considerable volume.

When, however, the bottom friction has produced the necessary retardation, the crest of the wave falls forward, as has already been explained, and

impact takes place at the precise stage at which the forward motion of the particles has become equal to the velocity of the wave, so that the stroke of the latter is delivered with maximum effect.

Taking all these diverse phenomena into consideration, it is evident that breaking waves result in the generation of four separate and distinct forces acting individually and collectively upon all obstacles and structures in their path.

(1) A direct horizontal force, exerting compression.

(2) A deflected vertical force, acting upwards and tending to shear off any projections beyond the face line of the obstacle, whether cliff or wall.

(3) A vertical downward force due to the collapse of the wave, exercising a particularly disturbing effect on mounds in shallow water and on beaches.

(4) The suction due to back-draught or after-tow. This also produces its most noticeable results on foundation beds, whether natural or artificial.

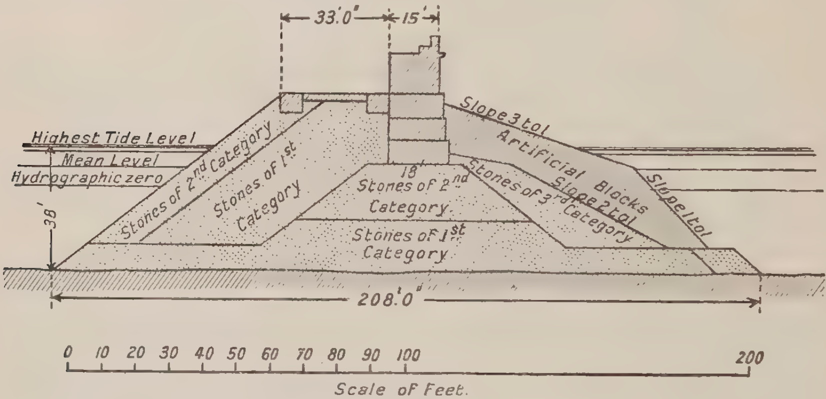


Fig. 122.—Section of Leixoes Breakwater.

Applying these fundamental facts to the question of breakwater design, it will be recognised that the phenomena to which they give rise are as follows :—

(1) A powerful momentary impact, combined with

(2) Hydrostatic pressure continuous for some short period, however minute, after the first shock.

Attending these principal effects there will be several subsidiary results, such as :—

(1) A vibration of the whole structure, tending to weaken the connection of the various parts.

(2) A series of impulses imparted to the water contained in the pores, joints, and interstices of the structure, producing internal pressures in various directions.

(3) The alternate condensation and expansion of the volumes of air which are confined in cavities and which may be unable to escape freely or in any way.

The exact determination of these stresses is practically impossible. Something, however, may be done towards estimating their scope and extent. How far this lies within the range of definite and effective calculation is our next concern.

The Dynamical Value of Wave Action.—The difficulties attending a determination of the precise effort of a wave are due to several causes. In the first place, there is the incompressibility of the water combined with the extreme mobility of its particles. Arrested suddenly in the course of motion, it produces all the percussive effects of a solid body in an infinite number of directions. No clearer evidence of this could be produced than the phenomenon known as water-hammer. If the outlet valve of a hydraulic service main be shut down abruptly, a blow is administered to the pipe which may be, and often is, sufficient to produce rupture, even at a considerable distance from the outlet, unless, as is generally the case, a relief valve is provided to prevent such a disaster.

In the second place, the wave stroke is both abrupt and continuous. Its first action is a blow, sharp and decisive, and of high momentary intensity.

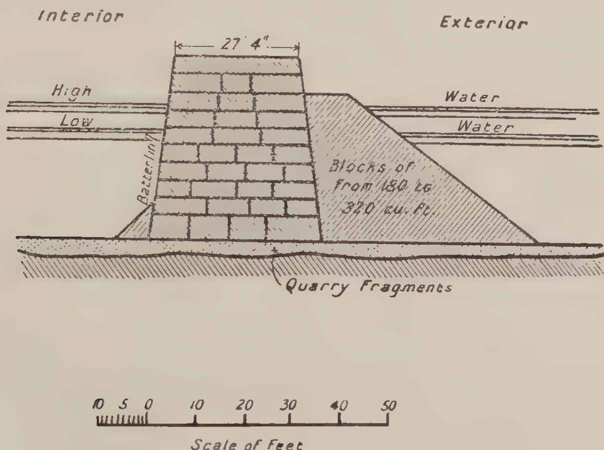


Fig. 123.—Section of Ymuiden Breakwater.

This is succeeded by statical pressure during the small but perceptible interval of time which suffices for the dispersal of the wave. Accordingly, there are two phases to be considered : (*a*) the initial concussion and (*b*) the subsequent pressure. Usually the question is dealt with entirely as a matter of simple continuous impact, but it should be noted that wave action is far from being completely identical with the unbroken impulse of a water jet.

Now, according to the principles of dynamics, the reaction of a surface subjected to continuous impact is measured by the rate at which momentum is destroyed. If, therefore, w be the weight of a unit volume of water, $\frac{w\,v}{g}$ is the mass which impinges on unit surface in unit time, and $\frac{w\,v^2}{g}$ is the rate

at which momentum is consumed. Hence, if p be the pressure on unit surface, we have

$$p = \frac{w\,v^2}{g}. \qquad \qquad (\alpha)$$

Accordingly, if it be possible to determine the velocity of impact, the pressure momentarily exerted may be deduced from this relationship.

Now, in order to determine the velocity, we have to take into consideration the depth of water in which the wave is moving.

In deep water—that is, when the depth of water exceeds the length of the wave—the speed is practically independent of the depth, and is almost exactly equal to the velocity acquired by a body falling freely through a height equal to one-half the radius of the circle, the circumference of which constitutes the length of the wave.[1] That is,

$$v_1 = \sqrt{2\,g \cdot \frac{l}{4\pi}}$$
$$= 2{\cdot}25\,\sqrt{l}. \qquad \qquad (\beta)$$

On the assumption that the wave is cycloidal in form, as in fig. 116, the height of the wave bears to the length the ratio $l = \pi\,h$. Accordingly we can write

$$v_1 = \sqrt{2\,g\,\frac{h}{4}}$$
$$= 4\,\sqrt{h}, \qquad \qquad (\gamma)$$

and substituting this value for v in equation (α) we get

$$p = 16\,\frac{w\,h}{g}$$
$$= \frac{w\,h}{2}; \qquad \qquad (\delta)$$

which leads to the conclusion that the pressure intensity of a wave in deep water is equal to the weight of a column of water one-half as high as the amount of free fall required to generate the specified velocity in the particles of which the wave is composed.

But the relationship $l = \pi\,h$ is an assumption not always, nor altogether, in accordance with observation.[2] For instance, as we have seen, on pp. 167, 170 *ante*, waves in the Atlantic have lengths up to 600 feet and heights of 40 feet

[1] Rankine, *Civil Engineering*, 18th edition, p. 754.

[2] Major Gaillard, of the U.S. Army, remarks that it is doubtful whether this ratio is obtained except under very unusual circumstances. Out of many hundreds of observations the nearest approach he found was $l = 5\,h$.

or so, in which case the ratio would be about $l = 15\,h^* = 4{\cdot}77\,\pi h$, giving a considerable increase in the estimated pressure. In view of the uncertainty attaching to the relationship between l and h, all we can conclude is that in deep water the impact of an unbroken wave tends to produce widely varying pressures which are, moreover, of relatively minor importance to the main subject of our enquiry.

In shallow water, the conditions are very different. The speed is no longer dependent on the length of the wave: it is identical for waves of all lengths, and varies with the square root of the depth. The velocity has, in fact, been determined to be nearly the same as that which would be acquired by a heavy body falling freely from rest through a height equal to the semi-depth of the water plus three-fourths of the height of the wave.[1]

In symbols

$$v_2 = \sqrt{2\,g\left(\frac{d}{2} + \frac{3\,h}{4}\right)}. \qquad (\varepsilon)$$

Let us give a series of values to d in terms of h and then incorporate the results in equation (a). From the consideration that d must not exceed l, and that when $d = l$, $l = \pi\,h$, the highest value theoretically assignable to d is $3\,h$.

$$\text{When } d = 3\,h,\ v_2 = \sqrt{2\,g\frac{9\,h}{4}} \text{ and } p = 4{\cdot}5\,w\,h$$

$$\text{,,} \quad = 2\,h,\ v_2 = \sqrt{2\,g\frac{7\,h}{4}} \quad \text{,,}\ p = 3{\cdot}5\,w\,h$$

$$\text{,,} \quad = \frac{3\,h}{2},\ v_2 = \sqrt{2\,g\frac{6\,h}{4}} \quad \text{,,}\ p = 3\,w\,h$$

$$\text{,,} \quad = h,\ v_2 = \sqrt{2\,g\frac{5\,h}{4}} \quad \text{,,}\ p = 2{\cdot}5\,w\,h.$$

The noteworthy feature about these results is that they give extremely high values for the pressure intensity, which is shown to be equivalent to the weight of columns of water ranging from $2\frac{1}{2}$ to $4\frac{1}{2}$ times the height required to generate the velocity in the particles. Pressure intensities so great must be accepted with reserve. They are much greater than any records which have actually been obtained.[2]

* The ratio is often much higher than this. Lieut. Paris, of the French Navy, found values as follows:—

In a light sea, $l = 39\,h$.
In a rough sea, $l = 21\,h$.
In a heavy sea, $l = 19\,h$.

[1] Rankine, *Civil Engineering*, 18th edition, p. 754.

[2] Major Gaillard, in his monograph on "Wave Action in Relation to Engineering Structures" (Professional Papers of the Corps of Engineers, U.S. Army, No. 31, 1904), already alluded to, interprets Rankine's statement rather differently, thus

$$v_2 = \sqrt{g\,(d + \tfrac{3}{4}\,h)}.$$

By another authority,[1] the velocity of waves in water, the depth of which does not exceed one-half of the wave length, is stated to vary only as the depth of water, without reference to the height of the wave. From the numerical example quoted, the relationship is deducible as:

$$v_2 = 5{\cdot}73\sqrt{d}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\zeta)$$

whence, in (α)

$$p = \frac{w}{g}(32{\cdot}8\,d) = \text{say } w\,d, \text{ approximately.}^2$$

Thus,

When $d = 3\,h,\ p = 3\,w\,h$
,, $= 2\,h,\ p = 2\,w\,h$
,, $= h,\quad p = w\,h.$

These values are substantially lower than the previous set, and more within the range of actual data. Indeed, for values of d corresponding to the depths of water in which waves are observed to break (say $\frac{3}{4}\,h$ to $\frac{3}{2}\,h$),[3] they are, if anything, an estimate on the low side.

The discrepancy in the results manifests the difficulty of assigning a definite numerical coefficient to the simplified equation, and all that is permissible is to write in general terms,

$$p = k\,w\,h,$$

where k has been given a fairly wide range of values. Thus

Dubuat gives $k = 1{\cdot}85$
Rayleigh ,, $k = 1{\cdot}96$
Mariotte ,, $k = 1{\cdot}25$
Gaillard ,, $k = 1{\cdot}31$
Jœssel ,, $k = 1{\cdot}62$
Thibault ,, $k = 1{\cdot}85$,

the average being 1·6.

The results obtained by this equation are somewhat less than those set out above. But, even so, they are found to be excessive. Major Gaillard states that he has applied the formula to several hundred observations upon shallow water waves, and almost invariably obtained results considerably higher than the observed velocities (page 107).

The exact working of Rankine is as follows:—"In water that is very shallow, compared with the length of the waves, the velocity is nearly independent of the length, and is nearly equal to that acquired by a heavy body falling through half the depth of water added to three-fourths of the height of a wave."—*Civil Engineering, loc. cit.*

[1] Darwin, on "The Tides," 3rd edition, p. 34.

[2] *Vide* equation (η) below.

[3] From observations on 58 waves varying from 4 inches to 6 feet in height just before breaking, Gaillard concluded that the critical value of d varied between $\cdot 72\,h$ and $2\,h$, and that for the greater number of observations d was equal to h. The average value of 134 observations on Lake Superior was 1·67, ranging between unity and 2·71. At the Firth of Forth, Stevenson found that $d = \cdot 72\,h$.

It is to be noted, however, that Gaillard considers the velocity of the crest of the wave, while breaking, to exceed the velocity of the body of the wave by 30 per cent., so that, if we give effect to this modification, his coefficient becomes raised to 2·33. This value, though much higher than any of the values of the other experimentalists quoted above, is not without support from the observations of Bidone, who obtained coefficients ranging from 1·5 to 2·3 for the pressure of water-jets.

The values assigned to k in the foregoing equation are all based on the assumption that the line of action of the wave is perpendicular to the surface on which it impinges. When the line of incidence makes an angle α with the surface, the coefficient undergoes further modification, and, according to Lord Rayleigh, k becomes

$$\frac{2\, g\, \pi \sin \alpha}{2 + g\, \pi \sin \alpha}.$$

When $\alpha = 90°$ it will be noticed that this expression coincides with the value of $k = 1·96$, given previously.

One point of interest about the fundamental equation $p = k\, w\, h$ is that it may be written

$$p = \frac{w}{2\, g} \times 2\, k\, g\, h;$$

and since w, the weight of salt water in lbs. per cubic foot, differs imperceptibly from the value of $2\, g$, the equation becomes practically

$$p = 2\, k\, g\, h; \quad . \quad . \quad . \quad . \quad . \quad . \quad (\eta)$$

or, giving k its mean value of, say, 1·6,

$$p = 3·2\, g\, h, \quad . \quad . \quad . \quad . \quad . \quad . \quad (\theta)$$

Based on the foregoing investigation, it is possible to construct a table of relative heights, velocities, and pressures for breaking waves, as follows :—

Height. Ft.	Velocity. Ft. per sec.	Pressure of Impact	
		In lbs. per sq. ft.	In tons per sq. ft.
10	22	1,024	·45
20	32	2,048	·91
30	39	3,072	1·37
40	45	4,096	1·83
50	51	5,120	2·29
60	55	6,144	2·74
70	60	7,168	3·20
80	64	8,192	3·65
90	68	9,216	4·11
100	71	10,240	4·58

The table must, of course, be taken only as a rough approximation, and it may be noted that the pressure in lbs. per square foot is about 100 times the height of the wave in feet.

So far, we have dealt only with the most obvious and notable effect due to the direct blow of the wave on the outer face of the breakwater. As, however, already noted, there are certain subsidiary effects entailing consequences of importance which must not be overlooked in breakwaters where joints and crevices afford admission of the wave to the interior. The principal of these are air compression and water-hammer.

Air Compression.—If a column of water impinge upon a volume of air imprisoned in a pipe, it can be demonstrated mathematically that the maximum internal pressure thus produced may amount to as much as $3\frac{1}{2}$ times the external pressure.[1] From this theoretical result must be deducted in practice the loss due to eddying action at the orifice and to friction. It would hardly be justifiable to rate these deductions at less than 30 to 50 per cent., but even so, we arrive at the conclusion that the maximum internal pressure may attain to nearly twice the external pressure. In an open joint or fissure in a wall breakwater, this pressure,[2] however local and of however short duration it may be, if constantly repeated, is bound to exert a most powerful disruptive influence. The desirability, therefore, of preserving an intact face to a sea wall is clearly apparent. Where the crevice, however, widens out into larger cavities (as in the case of a mound breakwater) the air compression is greatly reduced, and may even be entirely dissipated.

Water-Hammer.—The phenomenon of water-hammer is familiar in connection with water mains, and its results are only too obvious. It has been investigated in connection with hydraulic pipe lines by Prof. A. H Gibson, and some cognate effects as regards breakwater structures are described in a paper by the same author in the Minute of Proceedings of the Institution of Civil Engineers,[3] where it is affirmed that under conditions favourable to the production of water-hammer, and with very high velocities of impact, internal pressures ranging up to fifteen times the face pressure should be regarded as possible. Any internal air cavity, however, at the inner extremity of the joint would effectually prevent this hammer action, and as in the case of air compression, its influence, though extremely powerful, is momentary and local.

Measurement of Wave-stroke.—It is a matter for regret that few or no appliances are available for satisfactorily comparing the results of theoretical calculation with actual pressures. It is true that various kinds of apparatus have been contrived for the express purpose of registering the compressive

[1] Vide *Min. Proc. Inst. C.E.*, vol. clxxxvii., pp. 278, 279.

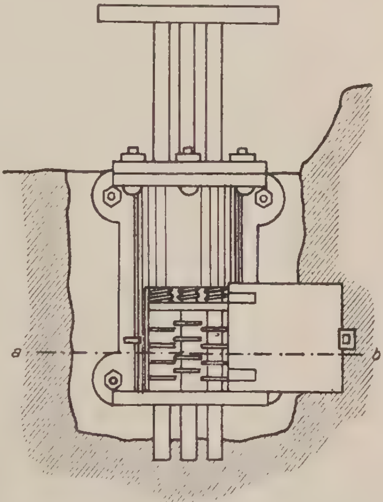
[2] There would be some further reduction in the case of a masonry conduit on account of the porosity of the material. This is difficult to estimate.

[3] Gibson on Wave Impact on Engineering Structures, *Min. Proc. Inst. C.E.*, vol. clxxxvii.

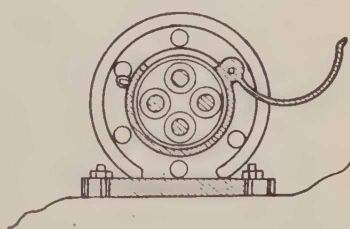
force of the wave-stroke, but for certain reasons these records cannot be considered an absolutely reliable criterion. The recoil of a spring is far from being a satisfactory method of gauging the colliding force of incompressible bodies. The very elasticity of the spring robs it of one of the most characteristic features of the ideal breakwater, and the retreat of the surface plate before the impulse of the wave is not in accordance with actual conditions. The real intensity of the blow, in fact, lies in the absence of yielding in either body. Theoretically, the effect of such impact is infinite, and in practice it must often far transcend the imperfect records of a none too sensitive spring dynamometer.

Furthermore, the assumption of uniform distribution of pressure involved in such means of measurement is untenable. Wave power is at least as subtle and irregular as wind pressure. Waves strike hardest in isolated places at uncertain intervals. The location of the dynamometer may or may not coincide with these places; in any case, it is a matter of mere hazard and surmise.

Yet, imperfectly as they realise ideal conditions, instruments of this class are as yet the only available means of obtaining practical data in regard to the force of waves. Stevenson's apparatus is perhaps the best known. It is illustrated in fig. 124. Several improvements have been contrived since it was first designed, but in principle it consists of a flat disc, perpendicular to which, and behind it, are arranged four rods passing through a firmly fixed cylinder. The disc is set fronting the sea, and when it is struck by a wave the rods are forced back simultaneously through the cylinder, thereby extending a spring connected with the front of the latter. On each rod is a leathern ring, which, prior to movement, is in contact with the back plate of the cylinder. The passage of the rods through this plate is unrestrained; but the rings cannot pass, and so they are forced along the rods. When the latter resume their original position under the recoil of the spring, the distance travelled by the rings is a measure of the intensity of the blow.



Plan.



Section.

Fig. 124.—Stevenson's Wave-stroke Dynamometer.

An instrument on these lines, but with special features, was constructed

a short time back by Messrs. W. H. Bailey & Co., Ltd., of Manchester, for use on the coast of Japan. It is illustrated in fig. 125. The principal modification consists in placing the instrument on trunnions with a swivel base-plate, so that it may be adjusted both horizontally and vertically to any desired angle. A pencil attached to the index-rod and a revolving drum, enable the record to be kept graphically over a continuous period.

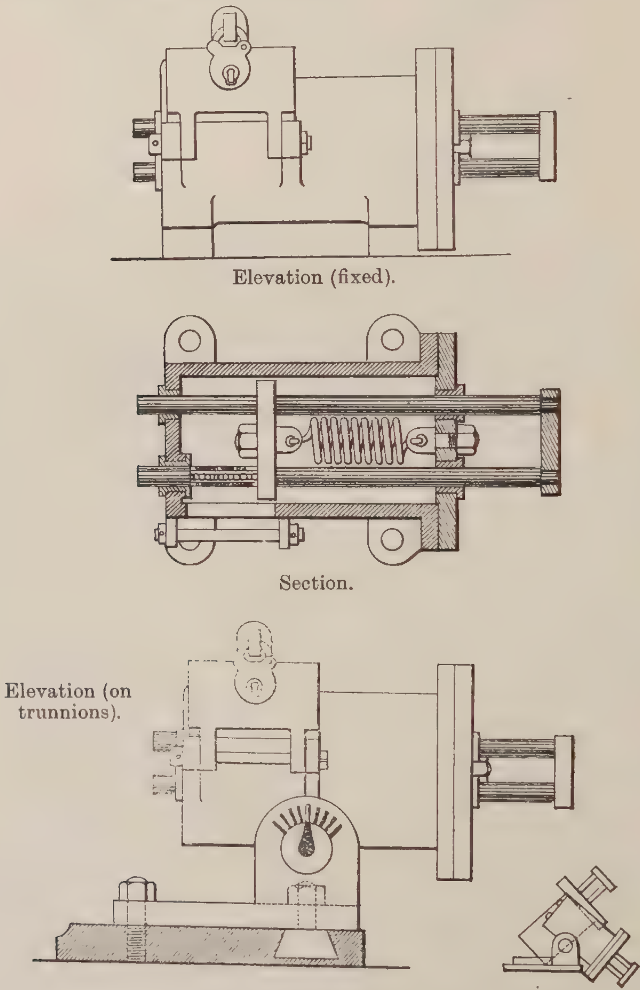


Fig. 125.—Bailey's Wave-stroke Dynamometer.

The calibration of these instruments is effected in the same way as ordinary spring balances--that is, by the imposition of dead loads. This method is open to the objection already stated, that statical pressure is quite a different thing from dynamical force, and a more appropriate system would be to calibrate by means of falling weights in units of kinetic energy. Yet,

even then, there would be the difficulty of the conversion of these last into their statical equivalents. No satisfactory solution has yet been put forward.

A dynamometer, lately devised by Major Gaillard of the Corps of Engineers of the U.S. Army, possesses a cylinder fitted with an elastic diaphragm and filled with liquid, which is in communication with a gauge. The mobility of the fluid particles enables the wave-stroke to be administered with less loss of energy than in the case of the solid plate, where the inertia of the moving parts has to be overcome. But the appliance is characterised by the same absence of conformity with actual conditions, to which attention has already been drawn.

The maximum pressure actually recorded by the marine dynamometer does not appear to have exceeded 3½ tons per square foot. At Skerryvore (in the Atlantic) a pressure of from 2½ to 2¾ tons per square foot has been observed; at Bell Rock (North Sea) 1½ tons; at Dunbar (East Lothian)

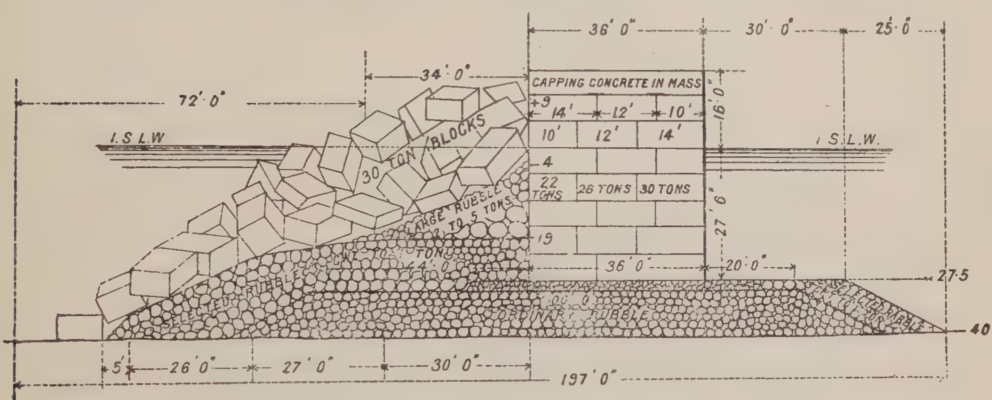


Fig. 126.—Section of Madras Breakwater.

3½ tons; and at Buckie (Banffshire) 3 tons. Stevenson further noted that the force of impact on a rising slope is six times as great as the pressure against a steep wall.

Some inferential records may be adduced in confirmation of the above. From experiments made with concrete blocks sliding upon a well-wetted concrete floor, Mr. Shield determined a frictional coefficient of ·7. In 1891, a section of the breakwater at Peterhead, weighing 3,300 tons in a single mass, was slewed bodily to the extent of 2 inches, without any dislocation of the substructure. The waves, therefore, must have exerted a pressure of over 2,310 tons, which, upon the surface exposed, works out to an average pressure of rather more than 2 tons per square foot.[1]

At Penzance, Mr. Frank Latham has noted pressures of from 18 to 20 cwts. per square foot. At Cherbourg, wave pressure is stated to vary from 5 to 7 cwts. per square foot.

[1] *Min. Proc. Inst. C.E.*, vol. cxxxviii., p. 400.

Examples of Wave Action.—Noteworthy instances of wave action are numerous, and many of them remarkable to a degree verging on the incredible. At Genoa, in 1898, blocks of artificial stone weighing 40 tons each are said to have been driven a distance of over 160 feet.[1] Blocks of 100 tons deposited in 1913 at Holyhead breakwater were found to have been disturbed by heavy seas, and it was decided to use in future blocks of 280 tons containing pig iron. At Wick, in 1872, a huge monolith weighing 1,350 tons is recorded as having been removed bodily from its seating, and deposited intact some

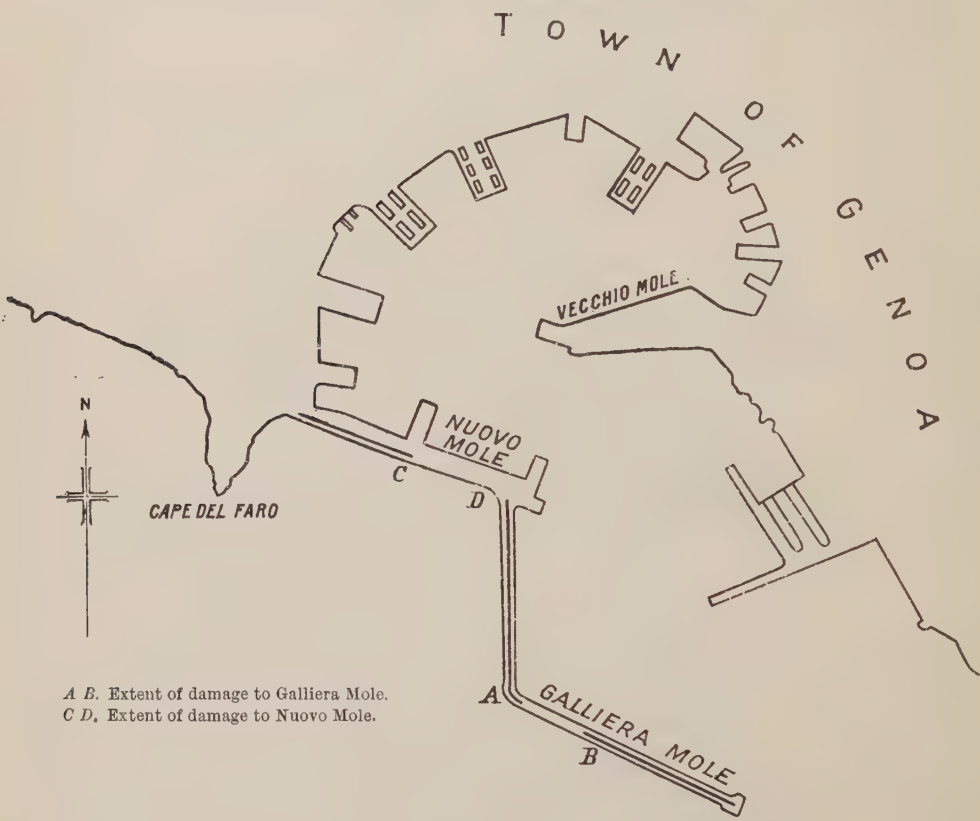


Fig. 127.—Harbour of Genoa.

considerable distance away. Another enormous mass of 2,600 tons at the same place was broken into two pieces and similarly displaced.[2] Yet another instance is afforded by the movement of the 3,300-ton mass at Peterhead, already alluded to. Movement, moreover, does not always take place in the direction of impact. Sometimes it is directly contrary to it. For instance, at Ymuiden, a face header-block weighing 7 tons was forced outward from the

[1] Bernardini on the Galliera Mole, *Proc. Int. Nav. Cong. Milan*, 1905.

[2] *Min. Proc. Inst. C.E.*, vol. xliii.

wall by the stroke of a wave which compressed the air in the joint at the rear.

Storm at Genoa.—It will not be without interest to consider, in some detail, the description of a great storm which damaged the breakwaters at Genoa in 1898. The following account is condensed from the report of Signor Bernardini to the International Maritime Congress at Milan in 1905.

The maximum fetch at Genoa is about 600 nautical miles, and the sector of exposure has an angle of 30° open to the south-west.

On the evening of 26th February, 1898, after several days of rainy weather and rough sea, both wind and waves became suddenly intensified. The gale augmented in violence as night advanced, reaching its maximum shortly after midnight. The wave crests, mounting higher and higher, finally leapt over the parapet of the Galliera mole and fell upon the inner quay. The light at the pierhead was visible until 3 a.m., when it suddenly went out. Although, as Signor Bernardini admits, it was easy to confuse the spray with the wave itself, and the grandeur of the scene was a temptation

Fig. 128.—Wave breaking over Galliera Mole.

towards exaggeration, it can certainly be said that the columns of water thrown up by the force of the waves, as they broke against the mole, attained a height of 65 feet during the early hours of the morning of 27th November. This is without taking into account a few small columns here and there, which rose to much greater heights—at least 100 feet. The height of the waves themselves is estimated from careful observation to have been about 25 feet.

It is curious to note that, contrary to what might have been expected, the atmospheric pressure did not fall in proportion to the unusual violence of the gale. In fact, the diagram recorded by the local hydrographic bureau indicates that the minimum pressure was attained on the night preceding this great storm, which exceeded all previous memorable storms in intensity.

The breakwater at Genoa is somewhat of the shape of a slightly distorted Z, and is divided into two sections, known as the Nuovo mole, adjacent to the shore, and the Galliera mole, further out. The Nuovo mole is 2,950 feet

long, while the Galliera mole comprises two arms 2,155 feet and 2,765 feet long respectively. In all, the breakwater is a mile and a half long. The damage wrought by the storm was as follows:—

Of the Nuovo mole a length of rather over 800 feet flanking the deepest

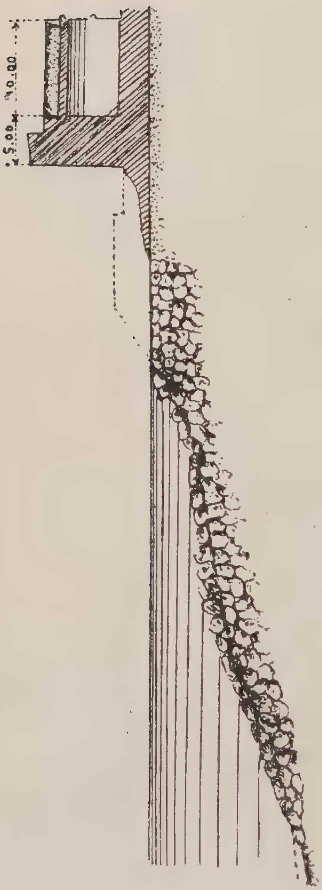


Fig. 129.—Part Section of Nuovo Mole.

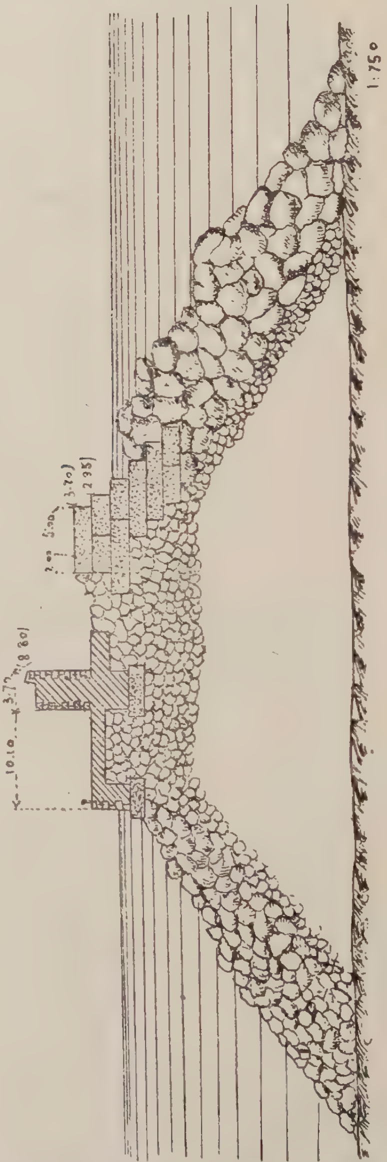


Fig. 130.—Section of Galliera Mole as constructed.

portion of the sea had its foundation laid bare, both the natural and artificial protection blocks being swept away and the front apron demolished, so that the foot of the wall lay exposed to the full effect of the waves. The nature

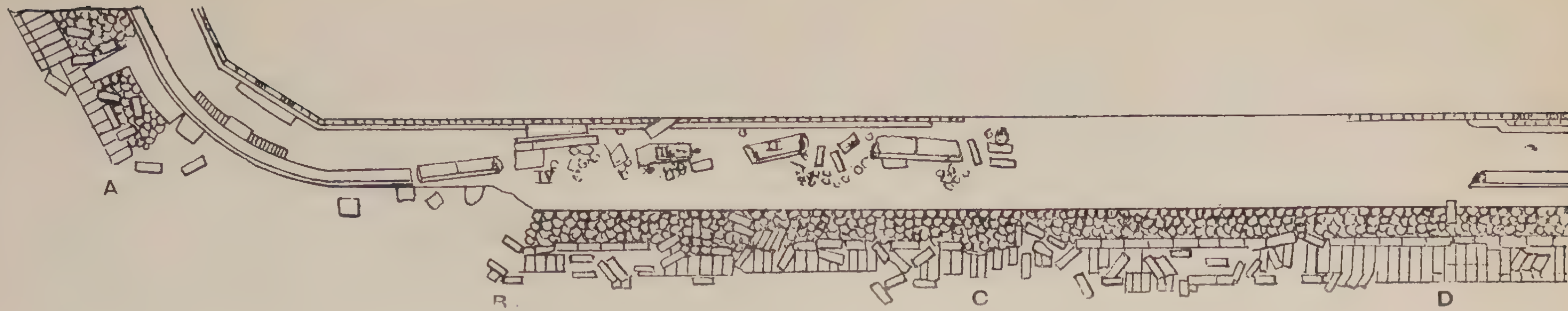


Fig. 131.—Plan of Portion of Galliera Mole showing extent of damage.

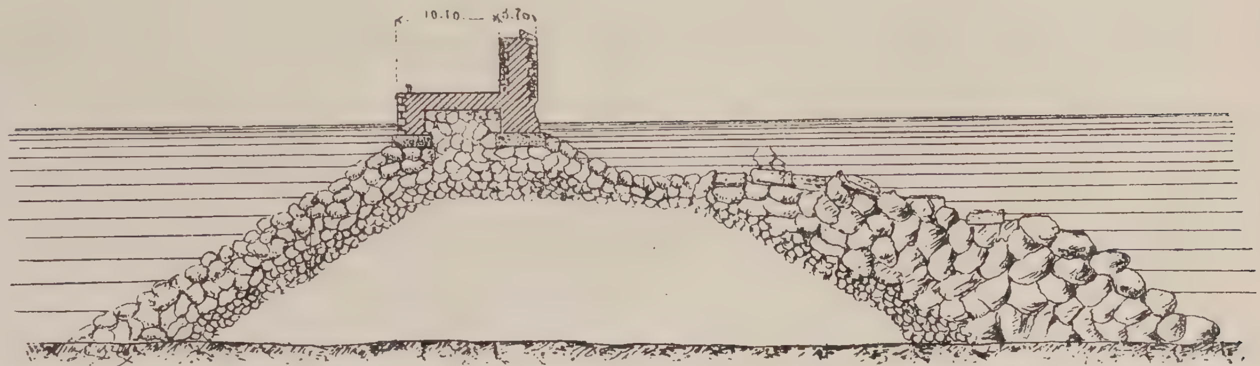


Fig. 132.—Section of Damaged Portion of Galliera Mole from A to B, fig. 131.

of this transformation is indicated in fig. 129, which shows the section of the mole both prior and subsequent to the storm.

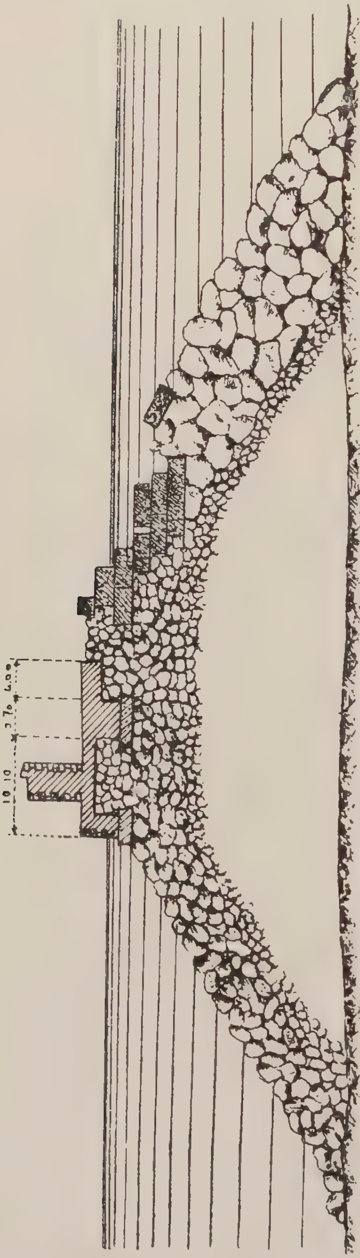


Fig. 133.—Section of Damaged Portion of Galliera Mole from B to C, fig. 131.

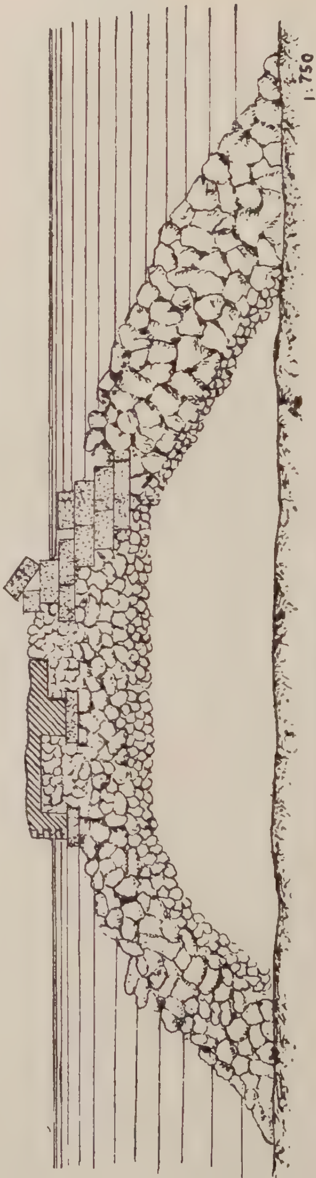


Fig. 134.—Section of Damaged Portion of Galliera Mole from C to D, fig. 131.

The Galliera mole had never suffered in any way from the frequent south-westerly gales to which it had been exposed during the ten years of its

existence preceding the storm of 1898. All that had been necessary was to renew from time to time the cement coating which had been applied as a preservative to the artificial blocks lying above water. When, therefore, the storm broke, every part of the mole was in perfect repair.

During the early hours of the morning of 27th November, despite the fact that the outline of the mole was almost completely obscured by foam and spray, the shelter wall of the outer arm was observed to be split into several sections for a distance of 500 feet from its junction with the inner arm. Some of these sections had merely shifted in position, but others had been completely overturned on to the inside quay. As the day advanced the breach was extended, until eventually it was 65 feet wide, the wall continuing to break away and small portions of it to be swept into the harbour. Fig. 131 is a plan of the damaged portion, and figs. 132, 133, and 134 are cross-sections at various points.

In the first length, AB, for a distance of about 230 feet from the commencement of the bend, the pitching or covering of artificial stone blocks was torn off to depths varying from 6 feet 6 inches to nearly 20 feet, the blocks in some cases being deposited along the outer slope, and in other cases projected a long way out on each side. The protecting apron was completely swept away, but the foundation of the mole structure was not damaged beyond a few cracks near the base, which were neither large nor deep.

In the length BC, the topmost course of artificial blocks of stone was overturned, and the lower courses were dislocated and partially damaged. The shelter wall was broken into five enormous blocks, of which four were shifted parallel to their original position and one (number IV. on plan) was overturned on to the quay. Detailed particulars of the blocks are as follows :—

No of Block on Plan.	Volume in Cubic Feet.	Weight in Tons.
I.	15,510	1,012
II.	6,980	455
III.	4,935	322
IV.	13,395	894
V.	14,030	915

In the length CD, the shelter wall was entirely demolished and the artificial blocks disturbed, though not to the extent experienced in the adjoining sections. One characteristic feature of the damage was the disruption effected by the air compressed within and below the masonry, which caused the latter to be projected upwards as if by explosion.

Several of the artificial blocks, laid as headers in the upper course, were displaced in a manner which deserves attention. They were found leaning against adjoining blocks, as if they had been acted upon simultaneously by two forces, one vertical and the other lateral.

Throughout the undamaged section of mole, the protection blocks were

set back about a yard. One bollard, struck by a portion of falling shelter wall, was sheared flush with the quay level. Several blocks of concrete, each weighing about 40 tons and having a volume of over 600 cubic feet, were driven a distance of 165 feet. This movement, of course, could not have been accomplished by any single stroke, but must have been the cumulative effect of repeated blows.

Storm at Bilbao.—Equally remarkable is the account of the damage wrought by a storm on the last day of the year 1894 at the port of Bilbao.

On that evening the action of the waves became so violent that the whole mass of protecting blocks covering the breakwater was completely carried away. These blocks had each a volume of $39\frac{1}{4}$ cubic yards and a weight of over 60 tons; they had been laid with the greatest care in contact with one another, forming an apron to the superstructure 26 feet by 16 feet, and consisting of two rows in width and depth alike. The toe of the superstructure being then unprotected, the latter work was soon undermined and demolished. The most striking feat of the storm, however, was the removal of a large monolithic mass of 1,046 cubic yards volume and 1,700 tons weight, placed at the extremity of the breakwater; it was carried a distance of 105 feet into the interior of the harbour.

These instances suffice to exhibit the vagaries which attend a demonstration of wave power by nature in her more violent moods. We pass on now to an application of these facts to breakwater design.

Classification of Breakwaters.—Practically all breakwaters fall within the limits of two types, the respective characteristics of which are

(1) The heap, or mound, and
(2) The wall.

The former of these is a heterogeneous assemblage of natural rubble, or undressed stone, in pieces of varying size, supplemented in many cases by artificial blocks of bulk larger than can be conveniently quarried in the natural state, the whole being deposited pell-mell, without any regard to bond or bedding.

The latter involves in whole, or mainly, the construction, in a regular and systematic manner, of a masonry or concrete wall, with vertical, or nearly vertical, faces.

Subsidiary classes form a series of gradations between these two distinctive types, so that strict lines of demarcation are not always easy to draw. The combination of wall and mound in varying proportions constitutes indeed by far the bulk of instances in modern practice. Sometimes the mound predominates and is simply capped by a slight superstructure of regular masonry, as at Algiers and Oran; in other cases, it is reduced to a minimum, becoming a mere foundation layer for a wall of massive and substantial proportions, such as is exemplified at Ymuiden and Zeebrugge.

The advantages and disadvantages attaching to each of the two principal

[To face p. 188.

Fig. 135.—View of Genoa Breakwater after Storm of 1898.

types may be considered under the heads of (1) Cost of Construction, (2) Cost of Maintenance, and (3) Efficiency.

Cost of Construction.—As regards the first point, much, of course, depends upon the locality of the breakwater and its coastal environment.

Where stone is plentiful and quarries lie conveniently adjacent to the site, the rubble mound will commend itself on account of the facility with which it can be formed, and the comparative economy resulting from the use of undressed stone, with its attendant unskilled labour. Such was the case at Portland, where there was not only an abundance of stone, but also a practically unlimited supply of convict labour.

In the absence, however, of these essential conditions, and provided the depth of water be not great and the foundation be sufficiently firm, the wall,

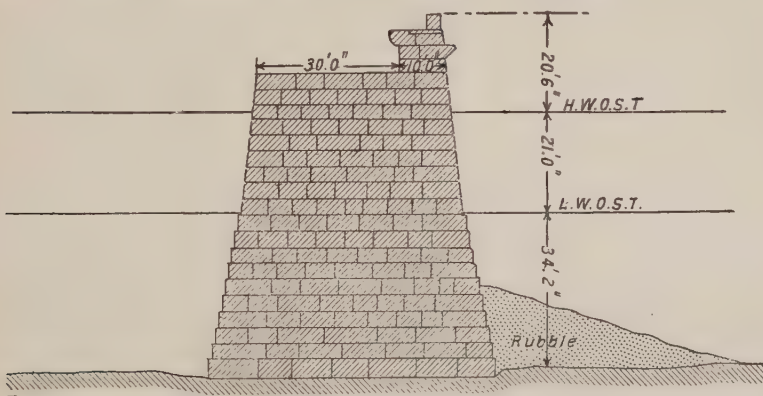


Fig. 136.—Section of Battery Pier, Douglas, I.O.M.

involving, as it does, a much less quantity of material, will be found preferable. Especially will this be the case where skilled labour happens to be plentiful and cheap. Even the difficulty of a defective foundation may be overcome by one or other of several expedients without perceptibly altering the relative positions. But where the sea bottom lies at a great depth the superior economy of the pure wall cannot be maintained.

Taking (merely for comparative purposes)[1] the cost of rubble stone at the quarry at, say, eighteen pence per ton, and allowing $1\frac{1}{3}$ to $1\frac{1}{2}$ tons to the cubic yard of stone, *in situ* (after deducting 20 to 30 per cent. for interstices), the total cost of obtaining and depositing a rubble mound under favourable conditions may be stated at from 3s. to 3s. 6d. per cubic yard of volume.[2]

[1] The European War has so profoundly affected the whole field of engineering expenditure that no estimate of more than transient value can be given. It is preferable, therefore, to adhere to pre-war prices in which such modifications can be introduced as are necessary to suit particular circumstances.

[2] The cost of the rubble mound at Holyhead ranged from 2s. 3d. to 2s. 7d. per ton, deposited in place. The cost of quarrying was 9d. per ton.

At Sandy Bay, U.S.A., 1s. 9d. per ton was paid for ordinary rubble deposited *in situ;* larger blocks, averaging 5 tons, were rated at 4s. 10d.

This, of course, applies to natural rubble deposited at random in the body of, and forming the bulk of, the breakwater. Larger and special material for protecting the surface slopes will run to 6s. and 7s. per cubic yard. Possibly 4s. or 5s. per cubic yard might be taken as an average all-round cost for the whole.

For dressed masonry or concrete work, either in the form of blocks or in bulk, set mainly below the water level, it is difficult to assess a rate without a full knowledge of the circumstances and resources at disposal; yet it would be unjustifiable to imagine the work as capable of being carried out at a lower rate than 2s. a cubic foot, and it might easily attain a very much higher figure.[1] Even this minimum rate is from eleven to thirteen times that of rubble work, so that, *ceteris paribus*, the bulk of the mound would have to exceed the bulk of the wall in something like the same ratio before it ceased to be the more economical method.

The cost of composite breakwaters combining a foundation mound with an upper wall will, of course, lie between both extremes, and probably, in the majority of cases, it will prove to be rather more than half the cost of an equivalent upright wall.

Actual examples affording any degree of serviceable comparison are difficult to quote, as so much depends upon the particular circumstances and conditions of each case. It would, in fact, be necessary to go very minutely into detail in order to estimate the relative value of each variation from its fundamental type, and, apart from this, no effective comparison could be made. All that can be said is that breakwaters have cost anything from £50 to £400 per lineal foot. The lower limit appertains to minor structures only. Among those of greater importance may be cited the following:—Portland breakwater cost approximately £130 per foot run; Holyhead, £160; Colombo, £170; Alderney, £235; Plymouth, £300; Peterhead, £300 [2]; and Dover, £370.[2] Other instances will be found in connection with their detailed descriptions.

Cost of Maintenance.—A comparison of the expenditure upon upkeep of the wall and the mound admits of only one conclusion.

The wall, provided it be carefully and properly constructed in the first instance, calls for no further attention save for such rare and occasional damage as results from some storm of exceptional severity.

The mound, on the other hand, is peculiarly susceptible to the constant fretting and attritional action of waves. Concussion and back-draught, or suction, constitute two alternating forces continuously and incessantly at work, even in times of moderate and calm weather. Rough rubble is smoothed and rounded by repeated movement, until it is easily sucked out of position

[1] Particulars of ashlar work at Holyhead breakwater: Runcorn sandstone below zero, 2s. 11d. per cubic foot. Anglesea limestone below zero, 3s. 5½d. per cubic foot. Runcorn limestone above zero, 1s. 9d. per cubic foot. Anglesea limestone above zero, 2s. 3½d. per cubic foot.

[2] Incomplete figures; estimates only.

and rolled away. The surface slopes thus become gradually less steep, while the flattening correspondingly increases the power of the waves, converting them more and more from the oscillatory into the translatory variety. The ultimate dispersal of a rubble mound left entirely to itself is only a matter of time. The preservation of a mound breakwater necessitates, therefore, a constant replenishment of material.[1]

The pitching of seaward slopes with ashlar work, or with massive concrete blocks, goes far to neutralise the destructive action; but the protection afforded is not always complete, and in cases where it has proved effectual, the result has only been attained by a much greater outlay than could justifiably be assigned to the formation of a simple rubble mound.

Efficiency.—The efficiency of a type is, after all, the consideration of greatest importance. Cheap construction and maintenance, though points to be carefully weighed, must inevitably be subservient to the attainment of the object in view.

The wall, rising up sheer from a sea bottom below the zone of disturbance with its exposed face vertical, or practically so, receives the wave before any conversion of oscillation into translation can take place. The wave is deflected upwards, and it falls back and down upon a bed of water too deep to permit of any deleterious influence upon the foundation.

On the rubble mound, with face slopes of 1, 2, 3, 4, and 5 to 1, the stroke of the converted wave is delivered with powerful and inimical effect—not only as regards the breakwater structure, but also the area which it incloses. The mass of water rushing up the seaward slope eventually falls over the crest, beating down upon the inner face and tending to effect a breach which must ultimately lead to serious results. Furthermore, even if the wave do not surmount the crest of the mound, the undulations of the sea are transmitted through the interstices of the stone mass and the harbourage area is kept more or less in a state of agitation. This action will be the more evident as the stones or blocks are of greater size, involving vacuities of corresponding magnitude. Amid large-sized artificial blocks deposited irregularly, the voids will amount to at least 25 or 30 per cent. of the whole; and as these blocks are employed to crown the majority of mound breakwaters, the protective value of the type falls considerably below that of a wall.[2]

[1] There is, as might be expceted, considerable variation in cost at different localities. The maintenance of the mound at Genoa is stated to be 7s. per linear foot per annum; at Naples 13s.; while, during a certain period, the Alderney breakwater involved an expenditure of from 25s. to 45s. This, however, was quite an abnormal experience. The renewal of large artificial blocks on the seaward slope of Cette forms an annual charge of 29s. per foot run. For a number of years subsequent to its construction the maintenance of Holyhead breakwater was as low as 1s. 3d. per lineal foot, but, in 1911, it was found necessary to put in hand a more extensive replacement of the wastage. The work was completed in 1914 at a cost of £60,000, and it involved the quarrying of 180,000 tons of rock from Holyhead mountain.

[2] At Marseilles it has been found that external waves 3 feet high give rise to fluctuations of from 4 to 6 inches within the area sheltered by the breakwater.

In order that a composite breakwater may possess the efficiency of the wall, it is necessary that its superstructure should commence at a depth of at least 5 fathoms, otherwise the back-draught of the waves will exercise an undermining influence upon the rubble foundation. The peculiar drawback attaching to this class of breakwaters is irregular settlement, whereby the superimposed wall is liable to be cracked and fissured. This point will receive further notice in the next chapter.

From the foregoing remarks, it will be seen that no absolute preference can be attached to any specific type of breakwater for general adoption. Questions of cost and maintenance, the degree of efficiency desired, the nature of the sea bottom, and the extent of exposure—these are all matters which have to be individually weighed before any definite decision can be arrived at. At the present time, there are breakwaters, either in course of construction or recently completed, of the pure wall type at Dover and Tynemouth, of the pure mound type at Brest and Marseilles, and of the composite type at Zeebrugge, Bilbao, and Peterhead.

We now enter upon a discussion of the conditions affecting the stability of breakwaters.

The Stability of Mounds.—It has already been pointed out that mounds are lacking in the quality of permanence. This applies more particularly to their upper portions which are under the constant influence of hydrodynamic action. The equilibrium of the lower portion is simply a question of quiescent hydrostatic pressure. Wave influence does not extend to an indefinite depth. Below the level at which its effects are felt, it has been found that rubble mounds will stand at slopes of 45 or 50 degrees.

The limiting depth of wave influence, however, is a matter of some uncertainty. It has generally been assumed, until recently, that a depth of 30 feet below the surface level marks the extreme boundary of the zone of appreciable disturbance ; but there are on record instances of serious wave action at greater depths. Thus at Peterhead Harbour, in October, 1898, blocks weighing upwards of 41 tons each were displaced by waves at a depth of 36½ feet below low water of ordinary spring tides. Instances of this nature, however, are very rare, and in the majority of cases the standard limit may still be counted upon as generally reliable.

The disturbing influence of waves is most keenly felt between the levels of high and low water, and it is in this region that the most trying ordeals of a breakwater are experienced. A difficulty underlying the situation is that in proportion as the slope is flattened to maintain its equilibrium, the disruptive effort of the wave is fostered and increased. Hence the introduction of huge blocks and monoliths to withstand impact. These blocks, which rarely weigh less than 25 or 30 tons a-piece, and often considerably more, may be deposited either in courses or at random. In the former case, they may be stepped so as to form a general inclination of 1 to 1 ; but if deposited at random, a flatter slope will be necessary.

The blocks, when artificial, are generally made in the form of rectangular solids: parallelopipeds in preference to cubes; and they should be laid as headers—that is, with their ends facing the line of wave action. In this way the minimum face area is exposed to the stroke, and there is the maximum resistance to overturning.

Natural blocks are heavier per unit volume than the majority of artificial blocks, and, for this reason, have claims to preference. They are also less

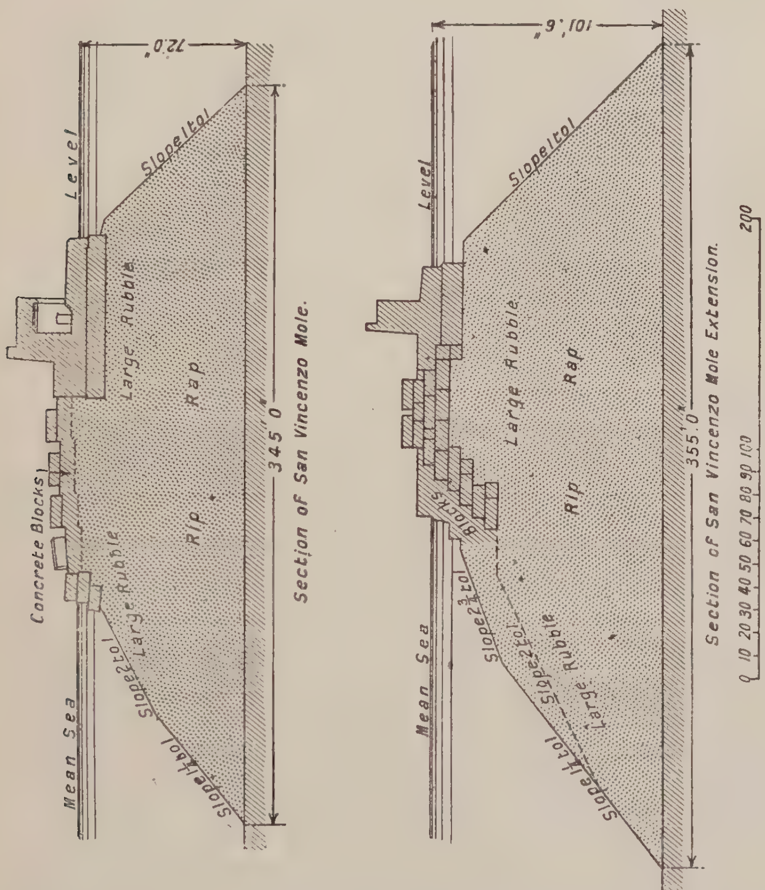


Fig. 137.—Sections of Breakwater at Naples.

liable to disintegration, but they are difficult to procure economically to large dimensions, and their irregular shapes render it impossible to bed them systematically. They have a tendency, also, towards becoming rounded like boulders, and this does not improve their steadiness *in situ*.

Mounds are most commonly formed in assorted layers, with the smaller material at the base and the largest at the top and on the flanks. Apart from the additional expense involved in selecting the material and of laying it in proper order, there is this further consideration, that such mounds are

less compact and less solid than mounds which are formed by an indiscriminate deposit of varied material, in which the smaller fragments occupy the interstices in the larger. On the other hand, a greater quantity of rubble is required for these cases.

The following statement of experience with mounds founded in the Tyrrhenian Sea is taken from a paper on Maritime Works in Italian Harbours, by Professor Luiggi, read before the School of Engineers in the University of Padua, 1909.[1]

1. Mounds of very small stone weighing ½ kg. to 2 kgs. (1·1 lb. to 4·4 lbs.) a-piece, deposited so that the crests attain to the level of 8 metres (26 feet) below sea level become, under the action of waves 6 metres (19½ feet) in height, inevitably worked down to a depth of 12 to 15 metres (40 to 50 feet) below sea level—that is, to a depth of two or two and a half times the height of the maximum wave breaking over them.

2. Mounds of stone in pieces varying from 2 to 50 kgs. (4½ lbs. to 1 cwt.) a-piece are better able to resist the shock of the waves, and may be deposited with crests reaching into the zone between 9 and 6 metres (30 feet and 20 feet) below sea level—*i.e.*, to a depth one or one and a half times the height of the wave—at which level the stones may be slightly disturbed, but are no longer transported or dispersed to any appreciable extent.

3. The subaqueous slopes spontaneously assumed by these mounds under wave action are about 2 or 3 of base to 1 of height. These slopes may be reduced to 2 to 1 on the outside of the mole and to 1½ to 1 on the harbour side, provided the surfaces are protected by blocks of large size, at least 500 kgs. (10 cwts.) each, and preferably 2 to 3 tons, ranging up to 10 tons.

Professor Luiggi adds, "We have in these statements of fact, now accepted without discussion by all maritime engineers, the elements for designing the subaqueous base of a mole in great depths of water, such as 25 to 28 metres (80 to 90 feet), as is the case at the Galliera mole at Genoa, and even 35 metres (115 feet), as at the mole of San Vincenzo at Naples."

Stresses in Wall Breakwaters.—We next turn our attention to the magnitude and extent of the disruptive forces acting upon upright walls so far as the stresses to which they give rise are measurable in numerical terms.

Considered as a structure exposed to the effects of wave action, a wall breakwater may fail partly, or wholly, in one or other of the following ways :—

(1) By the shearing of some bed-joint, or by the sliding of one component block upon another ;

(2) By overturning as a solid mass in sections of variable size ;

(3) By the uplifting and dislocation of a horizontal course, or layer ; and

(4) By fracture and shattering.

Assuming, for the sake of convenience and simplicity, that we are dealing

[1] Le Opere Marittime più adatte ai Porti Italiani : Conferenza letta il 22 marzo 1909 dall'Ing., Prof. Luigi Luiggi, Ispettore superiore del Genio civile.

with a single block or monolith of assigned dimensions, we may express the value of the adhesiveness of the bed-joint or base in its resistance to shear as from 3 to 5 tons per square foot if the cementing material be hydraulic mortar of good quality, and from 6 to 9 tons per square foot if Portland cement mortar, further assuming that, in each case, the proportion of sand to the matrix does not rise above 3 to 1.

Accordingly, the condition for equilibrium is that the horizontal component of wave pressure measured in tons shall not exceed the area of the bed-joint in square feet multiplied by some coefficient ranging from 3 to 9 in accordance with the nature of the cementing material of the joint.

This is in regard to shearing action. If the joint be already fractured, or if the adhesiveness be neglected, then resistance to movement can only be forthcoming through the agency of friction. The coefficient of friction has already been stated at ·7 (*vide* p. 181) for smooth concrete blocks, and a value of ·65 to ·7 will hold for all surfaces of masonry and brickwork in contact. For stone on rock the same value will suffice, but for brick or stone on moist, unctuous clay, the coefficient must be reduced as low as ·3. If the weight of a given block be W, then something like ·7 W is the force required to move it over a masonry or rocky surface, and from ·3 W to ·5 W over an earthen one.

A distinction must, however, be made as regards the weight of the block, whether it be submerged entirely, partially, or not at all. Substances immersed in water lose a part of their weight equivalent to the weight of the volume of water which they displace. Consequently, the effective weight of a completely immersed block is less than its weight in air by the weight of an equal volume of water, which, in the case of sea-water, it is customary to estimate at the rate of 64 lbs. per cubic foot.

The fact may be expressed in another form by stating that the weight of the block is equal to $(d - 1)$ times the weight of an equal volume of water, d being the density of the block compared with that of water as unity. This relationship has an important bearing on our next consideration.

Parenthetically, it may be pointed out that sliding action is very materially assisted by small smooth stones and pebbles more or less spherical in shape, which not infrequently intrude themselves between the detached blocks protecting the outer slopes of certain breakwaters.

The resistance of breakwaters or their component parts to overturning arises from their (effective) weight and from the tensional strength of the joints. This latter source should, however, not be counted upon. Beyond affording some slight additional margin of security, its assistance is so small as to be negligible, especially when compared with the inertia of the mass.

The **overturning effort** is due to the horizontal pressure of the wave, which exerts a moment about any point of the base measurable as $F\,x$, where x is the height above the base at which the effect of impact is assumed to be concentrated and F is the force of impact in statical units of pressure If the

block be small, and if its entire vertical surface encounter the full stroke of the wave, it is not unjustifiable to assume that the value of x is $\frac{h}{2}$ or the semi-height of the block. It is, of course, a matter of conjecture, but evidently it represents the extreme condition of things in an unfavourable sense, and, therefore, is a reliable basis of calculation.

But over surfaces of considerable extent the hypothesis of uniform intensity of pressure is not strictly tenable, and indeed, in certain cases, is very far from representing the actual effect of wave impact. The equivalent pressures at various points of an extensive surface are equally variable. The maximum occurs approximately at mean water level, and the force decreases above and below this point, in a ratio corresponding to the ordinates of some curve.

In the absence of complete and reliable data, the nature of this curve is more or less a matter of conjecture. It may be, and probably is, compounded of parabolic and hyperbolic segments. Such a curve has been deduced by Professor Luiggi[1] from data obtained in the Tyrrhenian and Ligurian Seas. Taking the wave as between 6 metres (19½ feet) and 7 metres (22¾ feet) in height, with jets of water up to 20 metres (65 feet) high, he assigns the following maximum values to wave pressure at the levels stated :—

From 15 metres (49 ft.) to 10 metres (32¼ feet) below sea level, ...	·2 ton per sq. ft.
As far as 6 metres (19½ feet) below sea level, .	·6 ,, ,,
,, 4 metres (13 feet) ,, .	·9 ,, ,,
At sea level, ...	2·8 tons ,,
As far as 4 metres (13 feet) above sea level, .	2·2 ,, ,,
,, 10 metres (32¼ feet) above sea level, .	1·9 ,, ,,

To what extent the ratio of values thus obtained is applicable with the same precision to other localities, *mutatis mutandis*, is, of course, open to discussion. It seems fairly well established, however, that the pressure at and about the sea level is a maximum, and, as we have seen on p. 181, it may be of very considerable intensity.

Now, the stability of a block is a function of $(d - 1)$ times the volume, for the moment of resistance to overturning is the product of the effective weight into a moiety of the width of the base. For critical equilibrium, therefore, we have—

$$F\,x = W\frac{b}{2}.$$

If, then, W varies as $(d - 1)$ V, it is noteworthy that any increase in d involves a much greater increase in W. Thus, if d be increased, say, from 2 to 3, the value of W is increased from V to 2 V, an increment, in the one

[1] Le Opere Marittime, p. 17.

case, of 50 per cent., and in the other of 100 per cent. Hence the great importance to be attached to the use, for sea work, of materials having a high specific gravity.

Although the influence of the bed-joint, in so far as it affords tensional resistance to the overturning action, is wisely neglected, on the other hand, it is not safe or desirable to ignore the effect of the corresponding compression upon the inner edge or line about which overturning may take place.

The resultant of the overturning force and the gravitation of the wall will often produce a very powerful and concentrated pressure upon a small area of the bed-joint, which may be beyond its capacity to resist. Thus, if the line of action of the resultant fall upon one or other of the two points which trisect the base, the intensity of pressure on the edge nearer the point is twice as great as the mean of the pressure over the whole area, and for any further eccentricity of the resultant, the ratio is greatly magnified. The following expression serves to convey a value for the intensity of pressure, p, on the nearer edge in terms of the eccentricity (x), the length (l) of the base-line, and the mean pressure (a) :—

$$p = a + \frac{6\,a\,x^{*}}{l}.$$

The maximum value of p consistent with safety is about 10 to 12 tons per square foot on Portland cement concrete, 8 to 10 tons on hard rock, 4 to 5 tons on rubble masonry, and from 2 to 3 tons on gravel, sand, or clay.

Uplifting is an action which takes place through the application of wave force to the underside of a mass. Obviously the dead (effective) weight of the mass is the resisting element, and the problem is a simple case of the equilibrium of two opposing forces, each of which has already been defined and described.

The **fracture,** or shattering, of a homogeneous block rarely results from the direct impact of the wave. When it does take place, it is probably caused by a prior dislocation, resulting in collision with other parts of the structure. The fracture of joints has been considered under the heading of resistance to shear. Blocks may also be fractured by unequal subsidence in the wall. This possibility applies more particularly to composite breakwaters, where the rubble foundation mound is subject to irregular settlement. The results can only be guarded against by avoiding the use of bond in the building of the wall, or by the adoption of what is termed "sloping" bond, as exemplified at Kurrachee and Colombo (p. 231).

Milan Conference; Report on Breakwaters.—The subject of Breakwater Design formed one of the topics of discussion at the International Maritime Congress of 1905. Papers, some of which have already been noticed, were presented by eminent engineers of various countries, and a general report was submitted to the Congress. This report was drawn up by

* See *Dock Engineering*, p. 176.

Professor lo Gatto, and a transcript of his conclusions cannot fail to be of interest. They were as follows :—

" Breakwaters built of rubble, although expensive in upkeep, are suitable for very sheltered sites in shallow water, provided good and cheap material is procurable. This type is not affected by the muddy or soft nature of the sea bottom.

" When the structure is exposed to very heavy seas, the rubble type of mole can still be adopted, under the conditions mentioned above, provided a revetment of concrete blocks is added outside down to a certain depth. The method of depositing these blocks at random appears the best as regards resistance and maintenance, on condition that the profile of the protected slope is so designed that it will shear the waves at sea-level. On the other hand, the method of setting the blocks in regular courses offers serious objections, as the blocks are liable to be disturbed by the settlement of the rubble base and to be completely destroyed during gales, and, in any case, they cannot be maintained in good condition without abandoning the principle of the system itself.

" Breakwaters with a rubble hearting and a double revetment of protecting blocks, laid in regular courses, are not at all reliable in very heavy seas, but they can render very useful service in sheltered sites and in waters of moderate depth, especially if the works are not of very great importance.

" Breakwaters with vertical, or almost vertical, sides are very suitable for moderate depths and hard sea-beds, where there is no fear of the undermining effect of the backwash and currents. They are very expensive and consequently inapplicable to unimportant works.

" The composite type of breakwater, consisting of a base formed by a loose rubble mound, surmounted by a vertical superstructure, is peculiarly suitable for tidal seas and for seas with a slight tidal rise and fall, provided the water in that case be very deep. In the case of tidal seas, there is no objection to stopping the superstructure at low water level.

" The type of construction in which the superstructure is made entirely of blocks laid in regular courses, can be adopted for seas which have but a slight tidal rise and fall, provided the site be sheltered. This type is not sufficiently reliable for heavy seas, and in some cases the system of large monolithic caissons can be adopted instead with advantage, on condition that the width of the blocks be suitably proportioned to their length, that the loose rubble of the hearting is perfectly compact, and that the very dangerous effects of the backwash at the seaward base of the blocks, which are produced by the impact of the waves, be counteracted by using exceptionally good material for the upper part of the apron or outside road on the sea face, or by loading and preserving this apron by means of protective blocks deposited at the base of the caissons."

The recommendations of this report formed the subject of some discussion and not a little adverse criticism on the part of the Congress in regard to

several of the opinions therein expressed. It was evident that unanimity could not be attained, and finally, the Congress limited the expression of its views to the following resolution :—

"The Congress refers to the information furnished by the written reports and oral observations ; it considers that engineers will find there information of great value for the construction of breakwaters, especially in regard to the force of waves, but, by reason of the great diversity of cases, it does not think that it should formulate any absolute conclusions."

With this summation of the special advantages and disadvantages attaching to the various types of breakwater exemplified at the present day, we bring our remarks on breakwater design to a close, simply adding some detailed reference to a few selected cases, chosen in illustration of the principles laid down in the preceding pages.

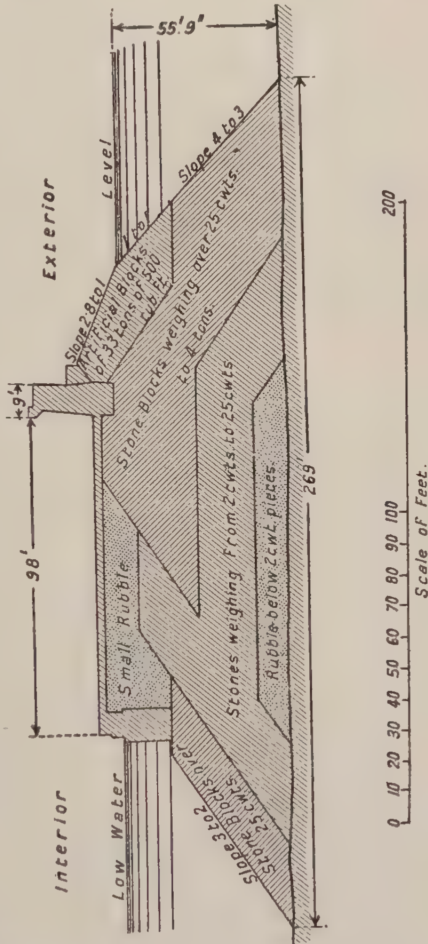


Fig. 138.—Section of Grand Jetée, Marseilles.

Breakwaters at Marseilles.—The main undertaking, begun in 1845, has a length, at the present time, of 4,530 yards, including an extension of 600 yards completed in 1904. The same principle of construction has been maintained throughout a period of sixty years with unvarying success.

A section of the breakwater is exhibited in fig. 138. The core is a bed of small rubble, having a depth or thickness of 10 feet, and lying upon the sea bottom at a depth of 55 feet below low water level. It is overlaid by layers of natural stone of increasing dimensions, ranging from 2 cwts. to nearly 4 tons a-piece. The quay shelter wall is a masonry structure founded upon the topmost layer of blocks.

The exterior slope is 4 to 3 for its lower portion, extending from the foundation to low water level. At this point it flattens abruptly to nearly 3 to 1. The effect of this sudden transition is to create a sharp ridge at the

water-line, with the result that the waves are cut at the point where their action is most potent. The upper part of a wave, therefore, falls dead upon the flat slope above, or, at the worst, upon the masonry apron in front of the shelter wall, in neither case capable of producing any deleterious results. The parapet thus receives no appreciable shock, and spray alone passes, at times, over its crest to fall upon the interior quay.

The blocks, forming the flattened slope referred to, are huge monoliths, rectangular in shape, with a length twice as great as their width, having a volume of 500 cubic feet and a weight of about 33 tons a-piece. They are deposited so as to lie longitudinally in the direction of the onset of the waves.

The external profile of the breakwater has proved to be extremely stable and is kept up at a very trifling expense in the way of repairs. For a length of 1,200 yards, constructed prior to 1865, the annual cost of maintenance is

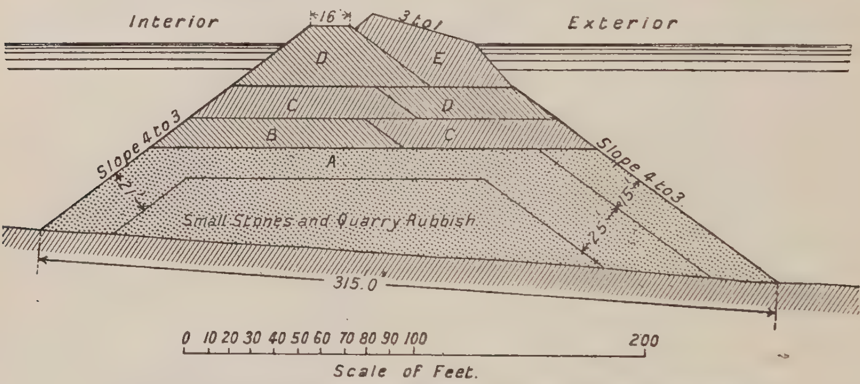


Fig. 139.—Section of Breakwater Extension, Grand Jetée, Marseilles.

A. Rubble stone up to 2 cwts. a-piece.
B. Natural blocks from 2 to 25 cwts. a-piece.
C. ,, 25 to 75 ,,
D. ,, over 75 ,,
E. Artificial blocks of 33 tons.

just under 1s. 9d. per lineal yard per annum. The remaining and later portion of the breakwater costs practically nothing for upkeep.

Apart from the parapet wall and the quay, the cost of the breakwater is stated by Baron Quinette de Rochemont[1] to have been as follows, according to the depth of water in which it was founded :—

In depths of 33 feet, £39 13s. per foot run.
,, 65 ,, 71 16s. ,,
,, 100 ,, 118 4s. ,,

According to M. de Joly,[2] however, the cost of the original breakwater, including the parapet and quay wall in a depth of 60 feet, was £127 per

[1] *Cours de Travaux Maritimes*, 1ère Partie, 1896.
[2] *Report on French Breakwaters* to Tenth Int. Nav. Cong., Milan, 1905.

linear foot, a figure which is evidently somewhat in excess of those quoted above, even when allowance is made for the additional work covered. M. de Joly's cost for the extension, however, is in accordance with Baron de Rochemont's figure for the same depth—viz., 100 feet.

Of its class, the *Grande Jetée* is an efficient example. Only one objection can be laid against the design, and that is the narrowness of the uppermost outer slope flanking the masonry apron. The existing width of 27 feet seems to be insufficient to prevent the protection blocks from being occasionally rolled off by the waves into deep water.

Settlements in the mass of the breakwater, though they have been by no means inconsiderable in themselves, appear not to have given rise to any serious dislocation of the parapet wall. Indeed, it is said that it is only possible to observe, on scrutiny, a few vertical cracks here and there, with widths of mere fractions of an inch. The shelter wall and its apron are not

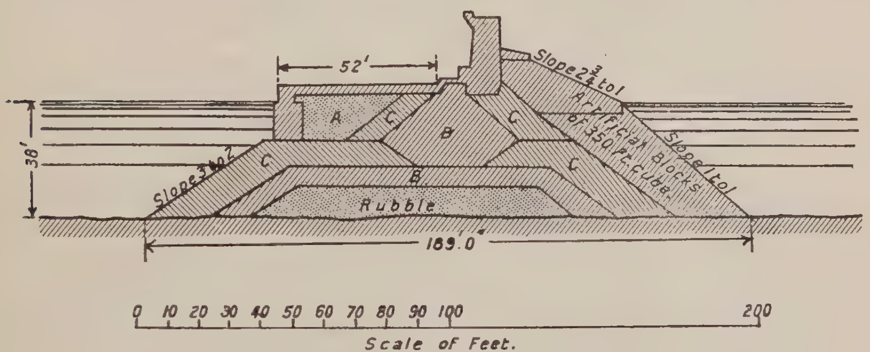


Fig. 140.—Digue de la Joliette, Marseilles.

A. Rubble deposited after construction of quay wall.
B. Stones from 2 to 25 cwts. a-piece.
C. Blocks above 25 „

bonded together: they are simply in contiguity. Separation was inevitable since they rest upon distinctly different foundations, the wall upon material of smaller size and greater compactness than the apron.

The *Joliette Digue* (fig. 140), constructed in about 38 feet of water, follows somewhat generally the same lines of construction as the *Grand Jettee*, in so far as regards the disposition of the material and the external slopes. The quay, however, is only about half the width of that in the previous case. The cost of this breakwater, including the outer protection blocks, amounted to nearly £154 per yard. The parapet wall with its ashlar work came to £61 10s. per yard, and the formation of the inner quay involved another £44, making the total cost, approximately, £260 per lineal yard.

Breakwaters at Algiers.—The principle underlying the design of the north and east breakwaters at Algiers, is that of approximating the rubble mound as closely as possible to the form and functions of an upright wall.

The mound has been laid to the very steep slope of 45 degrees throughout, and the superstructure occupies the whole of the narrow summit. Such a design is open to very strong and grave objections. The impulse of the wave, abruptly checked by the face of the parapet wall, is converted into a powerful downward force directed against the outer slope, or, alternatively, the waves, rising to an abnormal height, clear the parapet wall and break with considerable impact upon the inner side. In each case the tendency to disturbance is very pronounced, and movements frequently take place. Furthermore, the voids and interstices in the uppermost layer of rubble, which consists of stones of considerable size (some 500 cubic feet in volume), causes the external swell of the sea to be transmitted through the breakwater to the interior area, giving rise to an agitation of the surface, which is incompatible with efficient harbourage.

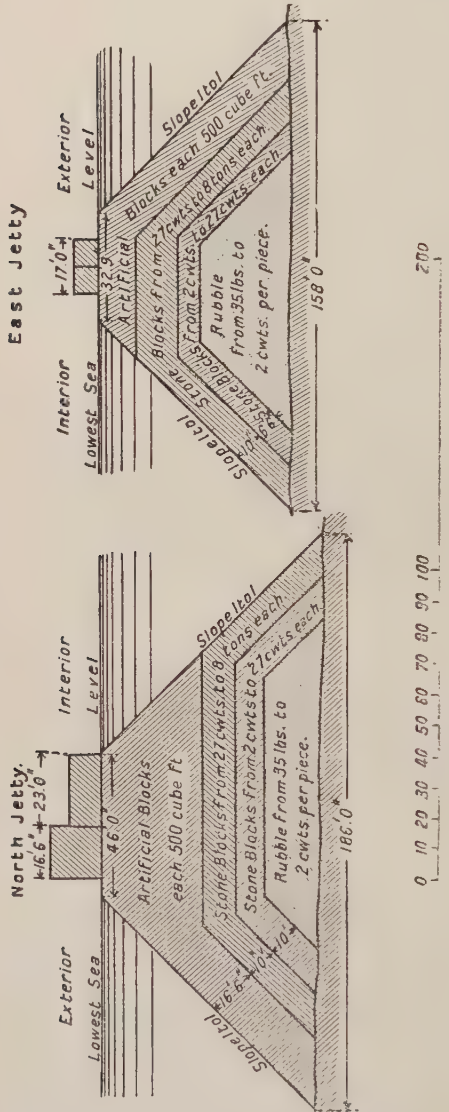


Fig. 141.—Sections of Breakwaters at Algiers.

Accordingly, in the construction of the inner port at the Agha, begun in 1899, the substructure of the jetty includes a solid wall of concrete blocks brought up from a depth of 18 feet below the water level. These blocks are not laid in bond—that is, breaking joint as at Genoa or Cette—on account of the highly compressible foundation, which is sand and mud. They are superimposed in such a manner as to form a series of piers, disconnected except for the masonry crown, which, though fairly continuous, is jointed every 25 or 30 feet. So far, the jetty has stood satisfactorily, but its construction is of too recent a date to admit of any definite pronouncement of its value. The cost is stated by M. de Joly to

amount to £350 per lineal yard, or about the same as the Marseilles breakwater extension, which, however, is of much greater sectional area and in water of much greater depth.

Breakwater at Sandy Bay, Mass., U.S.A.[1]—" The subject of an extensive harbour of refuge at Sandy Bay has been under consideration since 1882. The project submitted to Congress was for the construction, at a cost of $4,000,000, of a breakwater 9,000 feet long in the location shown on plan in fig. 9. The proposed breakwater was to be a rubble mound surmounted by a masonry superstructure founded 15 feet below low water. The mound was to be 40 feet wide at the top. The superstructure was to be trapezoidal in section, to rise 8 feet above high water, and to be 15 feet wide at the top. Below low water, it was to be laid ' dry ' ; above low water, in mortar."

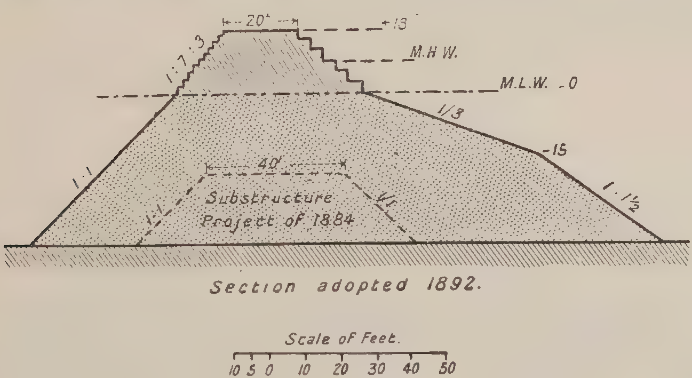


Fig. 142.—Section of Breakwater, Sandy Bay, U.S.A.

No work was ever done upon the superstructure above described, and, in fact, no project for the construction of a superstructure was adopted until 1892. In 1884 the plan for the substructure was changed to that of a mound 40 feet wide at the top, rising to 22 feet instead of 15 feet below low water.

The depth of water at mean low water varies from 6 feet at Avery's Ledge, the extreme southerly end of the breakwater, to about 89 feet at the extreme westerly end, and averages about 45 feet along the southerly arm and about 65 feet along the westerly arm. The bottom along the line of the work is nearly all ledge, except at the westerly end, where it is sand and shells. In the anchorage area, the holding-ground is excellent, being sand mixed with mud.

The work done prior to 1892, up to which time $450,000 had been appropriated, consisted in the placing of about 500,000 tons of stone in the substructure.

[1] McKinstry on Breakwaters, *Trans. Am. Soc. C.E.*, vol. liv.; Int. Eng. Cong., 1904.

In the early part of 1892, a board was appointed to recommend a project for the superstructure and any changes that might be desirable in the existing project for the substructure. The section adopted is shown in fig. 142.

By 1898, 600 feet at the northerly end of the southerly arm had been completed to full section, 1,200 feet more had been carried up to low water, and 2,800 feet more had been founded. The 600 feet of superstructure was formed of stones weighing not less than 4 tons each and averaging 6 tons, and the southerly 250 feet of it had settled some 2 feet. In the early part of the year, in a storm of exceptional severity, the 600 feet of completed superstructure was torn down to a height of about 5 feet above mean low water.

Modifications in design, recommended by a board of inquiry and adopted in September, 1902, are shown in fig. 143.

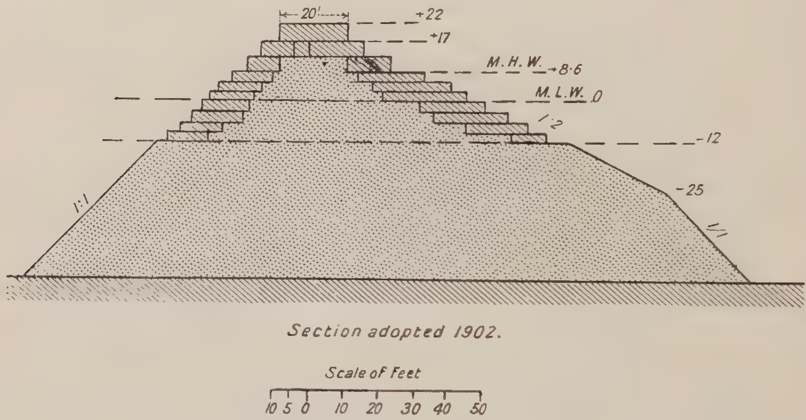


Fig. 143.—Section of Breakwater, Sandy Bay, U.S.A.

The capstones in the new plan weigh not less than 20 tons, are 20 feet long by 3 feet by 5 feet in end-section, laid on edge, and in as close contact as possible. The course below the capstones contains two stones, each weighing about 10 tons, the outer stone being at least 15 feet long and the inner one at least 10 feet. Below this course, to a depth of 12 feet below mean low water, the stones in the outer face weigh at least 8 tons, and in the inner face, at least 3 tons; and all are laid horizontally and as headers.

Including what has been spent up to 1902, the total estimate of cost for the work was $6,904,952. It is estimated that, when completed, the work will contain 6,301,407 short tons[1] of stone. To September, 1904, 1,690,178 tons had been deposited.

Piers at Tynemouth.—The original piers at Tynemouth, commenced in 1855, for the purpose of sheltering the entrance of the River Tyne, constitute

[1] Tons of 2,000 lbs.

an example of the composite type, in which the wall predominates. The piers (fig. 144) were built upon rubble mounds brought up to levels varying from 1 foot to 27 feet below low water in the case of the North Pier, and to 36 feet below low water in the case of the South Pier, the maximum depth

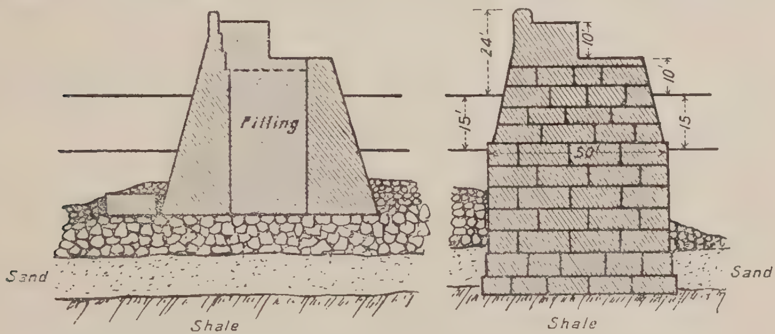
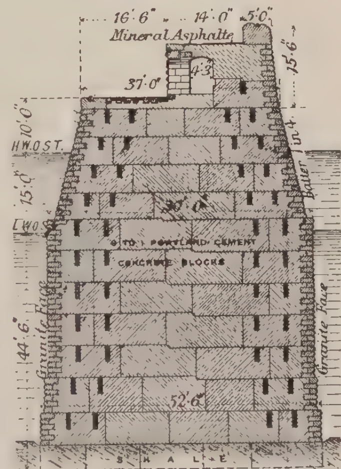


Fig. 144.—River Tyne Breakwater: Comparative Outline Sections of Old and New Work.

being, of course, in each case at the pierheads. The mounds were deposited on a bed of sand overlying, in turn, firm shale varied with boulder clay. The thickness of the sand bed did not, as a rule, exceed 10 feet. The mural portion of the piers consisted of two longitudinal masonry walls, connected at frequent intervals by cross walls, the cavities or pockets between being filled near the shoreward end with quarry débris, and further seaward with mass concrete.

In spite of the fact that it had so shallow a foundation—a defect which seems to have been realised even during construction, for to a limited extent where practicable the foundations were lowered—the North Pier stood well until the winter of 1893-4, at the end of which it was found that a length of 130 feet of the wall foundation was exposed and in some places undermined, although no actual settlement had taken place. As a temporary measure the hollow spaces were filled with concrete in bags, and bundles of scrap chain were deposited as a protection, until the foreshore blocks could be replaced. In the following spring



SECTION OF NEW STRUCTURE

Fig. 145.—North Pier at Tynemouth.

it was found that further damage to the foreshore blocks had taken place, and that some of the face-blocks and part of the foundation course of the sea wall had been drawn out. Repairs were put in hand, but the recurrence of gales before they could be completed nullified any benefit and produced

more damage. Notwithstanding every effort to restore the wall, in January, 1897, a length of 110 feet of the seaward face fell outwards, and in the following month a breach extending from side to side of the pier was caused. This breach became rapidly wider, and ultimately extended to a width of almost exactly 100 yards.

In consequence of so serious a mishap, the question of reconstruction was fully considered, and after careful deliberation, the Tyne Commissioners decided to form a length of new work within the line of the old work, as shown in fig. 146. This new wall is 1,500 feet in length.

Fig. 146.—Breakwater Piers at the mouth of the River Tyne, showing Reconstruction of North Pier.

The design which was adopted is shown in fig. 145. The foundation of the wall was laid directly on the shale at a depth of 44½ feet below low water, the old rubble having previously been removed. The lower portion of the work had a width of 50 feet, and the sides were carried up vertically to just above low water level. Above this there is a batter of 1 in 4 on each side reaching to 10 feet above high water level. The profile of the upper portion is similar to that of the old wall.

CHAPTER VIII.

BREAKWATER CONSTRUCTION.

Mound Construction—Barge System: Loading, Aligning, Discharging—Staging System: Erection and Maintenance, Trackways—Low-level System—Wall Construction: Staging and End-on Systems—Functions of Titans, Mammoths, and Goliaths—Caisson System—Foundations: Nature and Characteristics—Settlement—Wall Foundations—Piers—Piling—Limiting Loads—Surface Treatment—Levelling—Benching—Deposition of Concrete under Water—Bagwork—Block Making—Bond—Sloping Bond—Grouting—Minor Breakwaters—Crib and Box Work—Fascine Work—Examples of Breakwater Construction from Tynemouth, Alderney, Zeebrugge, Cette, Bilbao, Bizerta, and Dover.

Methods of breakwater construction are naturally as diverse as the local conditions which govern them, yet they fall, without undue constraint, under the same heads as those enumerated in our classification of the principles of breakwater design. Thus, we have the special methods appertaining to the formation of the mound and to the building of the wall. We will subdivide our observations accordingly.

Mound Construction.—For the purposes of a mound, no preliminary dredging operations are necessary. The material for the mound may be deposited upon the sea-bottom direct, for, from the very nature of things, it will spread itself sufficiently to distribute its weight within the limits of support, or it will sink until it reaches some firmer substratum by which the settlement becomes arrested. Nevertheless, it should be pointed out that dredging has not infrequently been resorted to when the surface of the sea floor is mud of a particularly impalpable character, and likely to prove treacherous. At Trieste, for instance, in consequence of certain mishaps, it was found necessary to remove a proportion of the softer mud. The rubble work did not subsequently sink so deeply as before, yet the settlement continued still to be considerable, amounting to 9 or 10 feet in depth. We shall have occasion later on to discuss more fully the question of settlement in foundations. Meanwhile, we are concerned solely with methods of construction.

Rubble may be deposited in one or other of three ways. These are:—

(1) By tipping or discharging from barges, scows, or other vessels afloat.

(2) By discharging from travelling gantries or from cranes, running on temporary overhead staging.

(3) By discharging from waggons passing over roads laid at or about the level of the top of the mound. The waggons are tipped in advance of the finished mound, upon which the roads are continuously extended as the work proceeds.

The Barge System.—The first method is best adapted to situations which are not unduly exposed. The difficulties of discharging from light craft in a rough sea must be sufficiently obvious. The system may, however, be practised with advantage wherever the meteorological conditions are favourable to the occurrence of reasonably long periods of settled sea.

The barges used are of two types: those having hoppers provided with bottom doors, which allow the stone to fall through, and decked barges, from which the stone is ejected overside. The former are available as long as the depth of water is sufficient to enable the hopper doors to be worked over the mound or place of deposit. For situations of minor importance a hopper barge of the ordinary type shown in fig. 147 may be utilised, but, obviously, it is only suitable for rubble of small size. For general use, barges of special design are necessary. Such barges are built without longitudinal keelsons. They have wells with vertical sides, and the doors are set at right angles to the longitudinal axis, thus giving large-sized openings through which the stone may be dropped. Pieces up to 5 tons in weight can be deposited by such barges. Figs. 148 and 149 illustrate the Lobnitz patent barge.

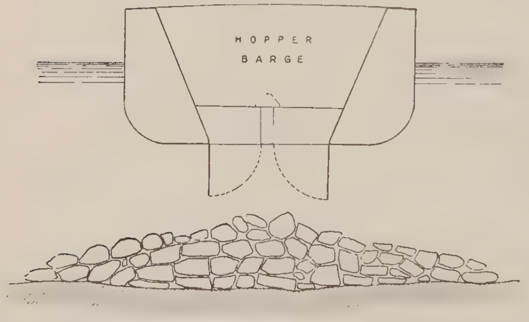


Fig. 147.—Hopper Barge discharging Load.

For work above the level at which hopper barges can be used, decked barges are resorted to. In the Mediterranean, these are of 30 to 50 tons burden, and they are loaded and towed upon an even keel. When moored over the site of deposit, water is admitted into one of the side compartments, causing the barge to cant over and the stone, which is piled on the deck, to fall overboard.

For depositing very large blocks, floating plant of heavy calibre has to be brought into requisition, and it is the practice to employ specially large floating cranes or sheers, not only for blockwork laid in regular courses, but also for wave breaker blocks, deposited more or less at random on the upper slopes of mounds. Such cranes and pontoons have lifting capacities of 20, 50, and even 100 tons.

One advantage attaching to the employment of barges is the opportunity afforded for depositing rubble uniformly and simultaneously over the whole site of a breakwater. This advantage is shared, but not to the same degree of freedom, by the staging method. With floating plant there is no restriction whatever, and the work may be prosecuted over a very extensive area without incurring any higher expenditure or any greater risk of misadventure. As will be seen when we come to deal with the question of settlement, this

consideration has a very important bearing on the permanence of breakwaters.

The loading of the barges is usually performed at the pier of an adjacent quarry by the ordinary means of tipping through a shoot, the stone being conveyed to the quay edge in waggons running on rails, the gauge of which is generally small. In the case of large blocks, cranes are necessary, both for loading and unloading. The loading crane is situated on the quay; the other is usually mounted on an attendant barge. A pair of sheer legs may take the place of a crane.

On arriving at its destination, each hopper barge, containing random rubble, is adjusted in position with the aid of suitable sight-lines fixed on the shore, or of any convenient landmarks. It is difficult to make satisfactory use of floating objects for this purpose, as they are necessarily moored in a flexible manner, and changes of tide and current may make sensible alterations in their positions, the exact extent of which depends, of course, upon the length of the moorings.

Satisfactory adjustment having been achieved, the barge load is discharged, either through the bottom doors or over side, as the case may be. It may then be necessary to trim the material, especially if it forms part of the upper layers. In tidal situations this may be done at periods of low water; otherwise, the services of divers are required. Care should be taken both by accurate alignment and judicious deposit to reduce the labour of trimming to a minimum, as it adds considerably to the cost of the undertaking. In shallow water the trimming and levelling of a rubble bed may be not unsatisfactorily achieved by supplementary hand-tipping, the inequalities in level being indicated by a sounding-lead.

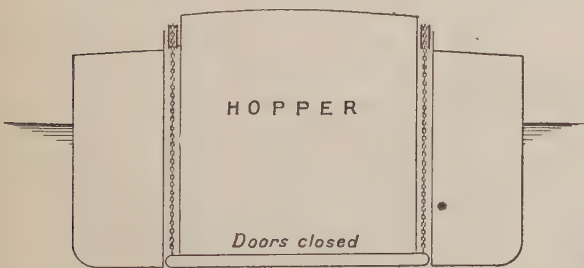


Fig. 148.—Lobnitz Hopper Barge.

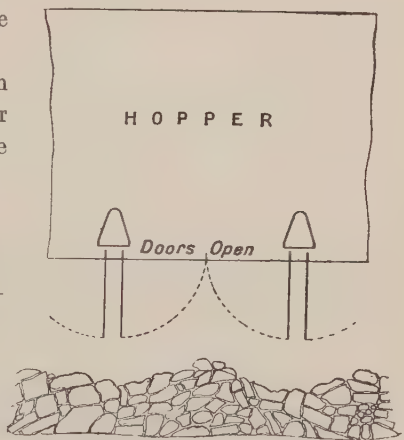


Fig. 149.—Lobnitz Hopper Barge.

Rubble should be evenly and systematically distributed over the entire width of base which the breakwater is intended to occupy, as also, where possible, over the entire length. Broken ridges and isolated heaps of stone give rise to currents and so to scouring; and although any effects of this action may be rectified by subsequent deposits, yet an additional supply of

material is entailed, as well as loss of time and labour. At Cette, excavations ranging from 3 to 5 feet in depth were found to have been produced by scour alongside rubble deposits which had been irregularly made.

The Staging System.—The use of staging, though primarily more expensive than any other method of procedure, is attended by many direct and indirect benefits. It promotes, to a very great extent, the unbroken sequence of operations, which is, perhaps, the highest desideratum from every point of view, and it affords greater protection to those engaged upon those operations than, at any rate, can be guaranteed by the barge system. Interruptions of more than a few hours' duration during tempestuous weather occur but rarely, and there is little time lost in waiting for the subsidence of the sea after a storm has spent its force. Of course, this assumes that the staging itself suffers no appreciable damage. It cannot be denied that temporary structures of slender build exposed to the full force of a gale run some risk of destruction—partial, if not complete. Collapses of greater or less extent have proved this beyond question, yet the instances are not so numerous as to warrant the attachment of very serious importance to the objection, and the particulars are not infrequently exaggerated. Thus, writing in 1904, Sir William Matthews, K.C.M.G., said:—

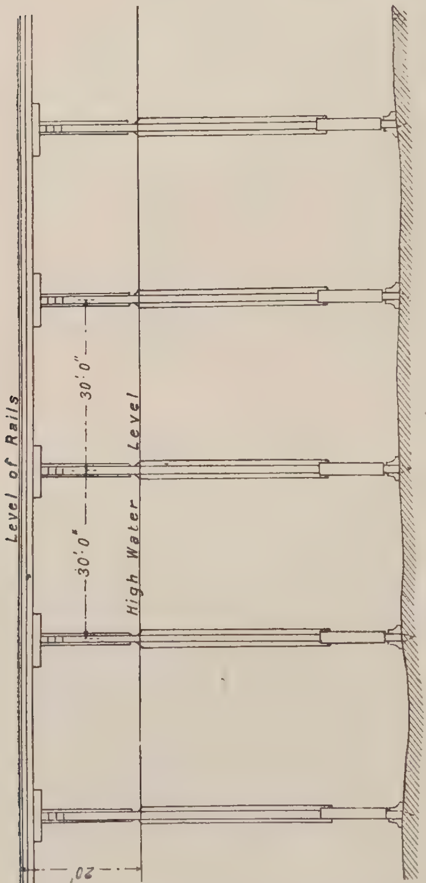


Fig. 150.—Longitudinal Elevation of Staging, Holyhead Breakwater.

"Notwithstanding the alarming reports which have appeared in the press, from time to time, with regard to the works at Dover, it is satisfactory to state that practically no damage whatever, from the first, has been occasioned to the permanent works, and only comparatively insignificant damage, having regard to the magnitude of the undertaking, has been caused to the temporary structures. Although it was alleged that during the great gale in September, 1903, one thousand feet of breakwater works and staging had been carried away, the only loss which was occasioned was the turning over of one span of

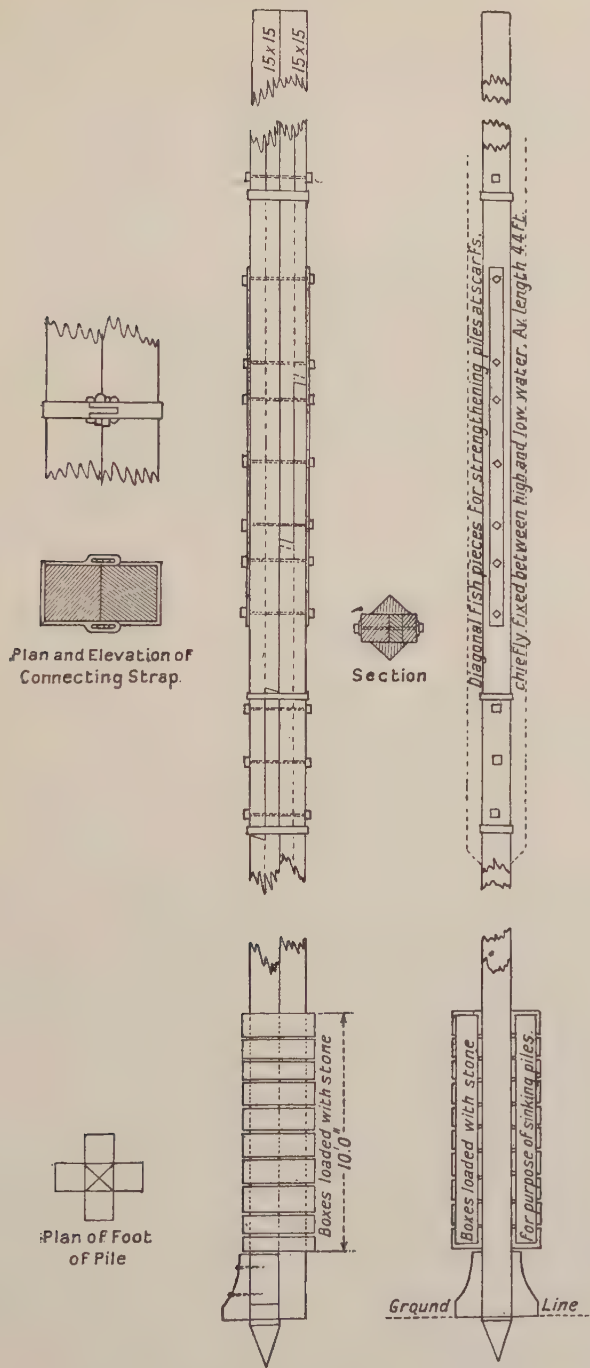


Fig. 151.—Staging Piles, Holyhead Breakwater.

temporary staging of 50 feet in length with the plant thereon, which, at that time, occupied an isolated position." [1]

A more serious source of danger to sea staging is from attack by marine organisms, and it is the more to be feared in that their depredations may remain undetected for some time. Constant inspection, therefore, is absolutely essential, and there can be no feeling of security. We have dealt however, with this matter more at length in a previous section.

Apart from these drawbacks, staging forms a steadier base for working purposes than a barge or vessel. Platforms may be affixed to it, or suspended from it at any desired level. It acts as an aid to the alignment of the work, and it allows appliances of a more powerful and efficient character than

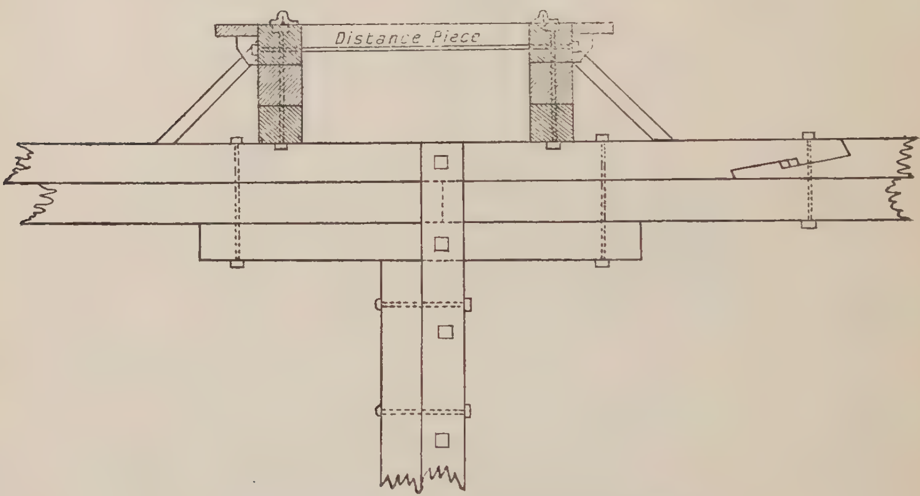


Fig. 152.—Rail Track, Holyhead Breakwater Staging.

floating plant to be employed. Some of these features are equally characteristic of the third or low-level system, but we shall see that there are also corresponding defects in the latter which are not applicable to staging.

Staging, as usually practised, takes the form of a series of piles in one or more rows of double line, driven at regular intervals (say from 15 to 50 feet apart) and connected by longitudinal runners (or, for the longer spans, by wrought-iron or steel girders), and bracing, with side strutting, cross bearers, etc. It thus forms a track, or a number of parallel tracks, for waggons, travelling gantries, and cranes ; and, in order that as little surface as possible may be presented to wave action, these roads or tracks should be located well above the highest sea level, say not less than 20 feet, and preferably 5 or 10 feet more, though, of course, any increase in the height is made at the expense of stability. Furthermore, the solid strutting, which characterises much land staging, is best replaced by slender tension members—chains and

[1] Matthews on Harbours of Great Britain, *Trans Am. Soc. C.E.*, vol. liv.

wire rope stays attached to secure moorings; or, if the surging of these under wave action be deemed undesirable, second-hand railway metals will be found eminently useful for the purpose.

The piles, wherever possible, are driven into the ground by a pile-driver rigged up on a barge or floating platform, or supported on a carriage which projects over from the land or from the staging previously completed. The floating pile-driver (or rather, a number of such appliances) can, in still water, construct the road at a much quicker rate than the stage pile-driver, which is limited in the scope of its operations.

In ordinary firm ground, the above is the usual course. If the ground, however, be of a very soft and yielding character, it will be desirable to substitute screw piles with a broad-bladed screw at the foot to afford the necessary surface bearing, or the pile may simply be set upright upon a large iron base-plate in the form of a shoe. A very broad and fairly thick stone slab, carefully set upon the surface of the ground, will often afford a sufficiently substantial base. This method, however, entails much cross strutting between the piles. Finally, if the sea bottom be rocky, the lower ends may be let into sockets drilled in the rock and steadied by concrete filling, or the pile may be shod with a stout iron spike, capable of being driven several inches, at least, into the solid.

For rubble work, the tracks are such as will suit the waggons in which the material is conveyed. The number of these tracks and their distances apart will depend upon the actual extent of the breakwater, but 25 feet or so seems to constitute a fairly average distance between track centres, and there are few breakwaters where the number of such tracks need exceed six, affording a width of 175 feet over all.

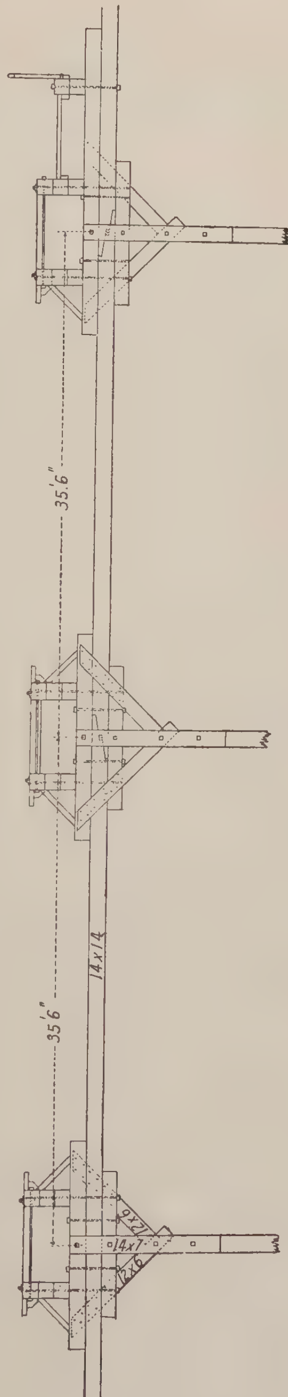


Fig. 153.—Transverse View of Staging, Holyhead Breakwater.

The stone, having been conveyed directly on to the stage in waggons, is tipped either by hand or by automatic arrangement, the waggons being tilted at the ends or at the sides. The staging method is particularly convenient on account of its adaptation to an organised continuous supply of stone, and the ease with which waggons may be marshalled and discharged. But it does not command the same extent of area as the barge system, unless the staging be erected from end to end in the first instance, which is unlikely, owing to the delay, risk, and cost.

Under general circumstances, staging may be utilised several times over in different positions; in other words, it is not necessary to provide for a length of staging equivalent to that of the breakwater. As the work is completed, the rear staging may be moved forward, connection with the ground level being maintained by sloping ways. There is inevitably some interruption while the change is being made, and any marked restriction of the working length tends to cripple progress and increase the cost.

There is one serious drawback connected with the use of the staging system which should not be overlooked, and this arises from the inability to deposit the stone with a slope sufficiently flat to suit the seas which may be expected in winter. Stone tipped from waggons into water does not in the first instance form a slope flatter than $1\frac{1}{4}$ to 1; frequently the stone will stand at less than 1 to 1. Such a slope is, of course, at once drawn down when heavy seas come on, but the process of forming a flat enough slope for any particular locality may occupy several winter seasons.

Another point to be borne in mind as constituting a difficulty where temporary staging is employed, is the cost and time absorbed in withdrawing the piles from the completed portion of the mound. Assuming the breakwater to be 30 to 40 feet deep, and the piles to have been driven at the outset of operations some 10 or 15 feet into the ground, it is obvious that fairly heavy tackle will be required to withdraw the piles after the mound is tipped.

Low-level System.—The low-level system of tipping, by means of waggons running along tracks laid on the solid breakwater structure as it advances, saves the cost of staging and reduces the amount of work done during the process of discharge; but, at the same time, it is a method which is greatly restricted in scope, in that operations are limited to the immediate vicinity of the completed mound; for, until the bank be raised at least above sea level, any extension of the railway track is impracticable. And allowing, under the most favourable circumstances, that the lines can be laid close to the water level, there is the added risk, the probability, the certainty even, that rough weather will cause frequent irruptions and settlements, so that there will be more or less constant relaying of the tracks—all entailing delay and expense. On the other hand, it may be urged that the use of the completed section of the mound as a roadway for the transport of materials assists to consolidate it, and to reveal any sources of weakness which it may ossess. This is no doubt true; but whether it affords sufficient justification

for the adoption of a system, which is otherwise slow and restricted, is a point which must be determined by the special circumstances of the undertaking. It is a form of construction which is not generally suitable for works on a large scale. For small embankments, however, it may be considered convenient and economical if time be not a matter of importance, and it produces substantial and reliable work.

Leaving the mound type of breakwater at this point, we pass on to methods of wall construction.

Wall Construction.—The masonry wall, built with prepared blocks of ashlar or concrete, carefully bedded and laid in accurate alignment, manifestly calls for more elaborate and less rudimentary appliances than are available for the formation of mounds. Other kinds of wall, such as those consisting of concrete deposited in mass in a fluid condition,[1] or built up of sacks and bags laid in courses, also demand special apparatus. The methods of construction generally adopted may be ranged under the headings of :—

(1) The Floating Plant System.
(2) The Staging System.
(3) The End-on, or Over-end, Low-level System.
(4) The Caisson or Buoyant Monolith System.

The Floating Plant System.—The use of floating plant of high lifting power for depositing large blocks of stone or concrete at random on mounds has already been alluded to. The same plant can be utilised for setting blockwork in walls, and in the Mediterranean it is very commonly employed for constructing the combined type of breakwater-quay. Artificial blocks up to 100 tons in weight are readily manipulated, and can be deposited at the rate of from 20 to 30 blocks per day under favourable conditions.[2] It is practicable, of course, to employ appliances even more powerful than these, and, although not strictly coming under the head of breakwaters, the method of constructing a quay wall at Dublin may be alluded to, in which blocks consisting of 3,000 cubic feet of masonry and weighing 350 tons each, were set in position by means of floating sheers. Advantage was taken of the buoyancy of water to relieve the pressure on the lifting tackle by 80 to 100 tons, the blocks being submerged to one-half their height.[3] But, even with this deduction, the dead weight handled was as much as 250 tons.

Floating plant, however, labours under a serious disability, in that it is subject to restrictions imposed by the vicissitudes in the weather. Floating derricks engaged on block setting require a perfectly smooth sea : whenever the surface is in the least disturbed they are obliged to suspend work, whereas for land plant the conditions may be quite favourable. In the Mediterranean

[1] The deposition of concrete in a plastic or partially-set condition is a practice to be deprecated.

[2] Luiggi on Le Opere Marittime più adatti al porti Italiani, 1909.

[3] Stoney on the Construction of Harbours and Marine Works with Artificial Blocks of Large Size. *Min. Proc. Inst. C.E.*, vol. xxxvii.

there are on an average about 100 days per annum on which operations can be carried out afloat, as against some 200 days available for a Titan ashore. The loss occasioned by the delay and the accumulation of capital charges is, therefore, anything but negligible.

The Staging System.—As regards the staging system, there is little to add to what has already been written in connection with mounds. The same lines of formation are followed and the process of depositing is the same, with the exception that, instead of being tipped in bulk, each block of stone is laid individually in position. Cranes or gantries are, therefore, an integral part of the system, and the tracks will be arranged to suit their requirements. For concrete work, platforms may either be erected on the staging itself, where the materials can be incorporated and discharged into shoots conveying it to its destined situation, or mixing may take place in the yard ashore, and the concrete be conveyed in skips to its appointed place. The former method has the advantage of greater convenience of output, the concrete machines being allocable in various parts, so as to command an extensive range, and there being no tendency to block the service-lines.

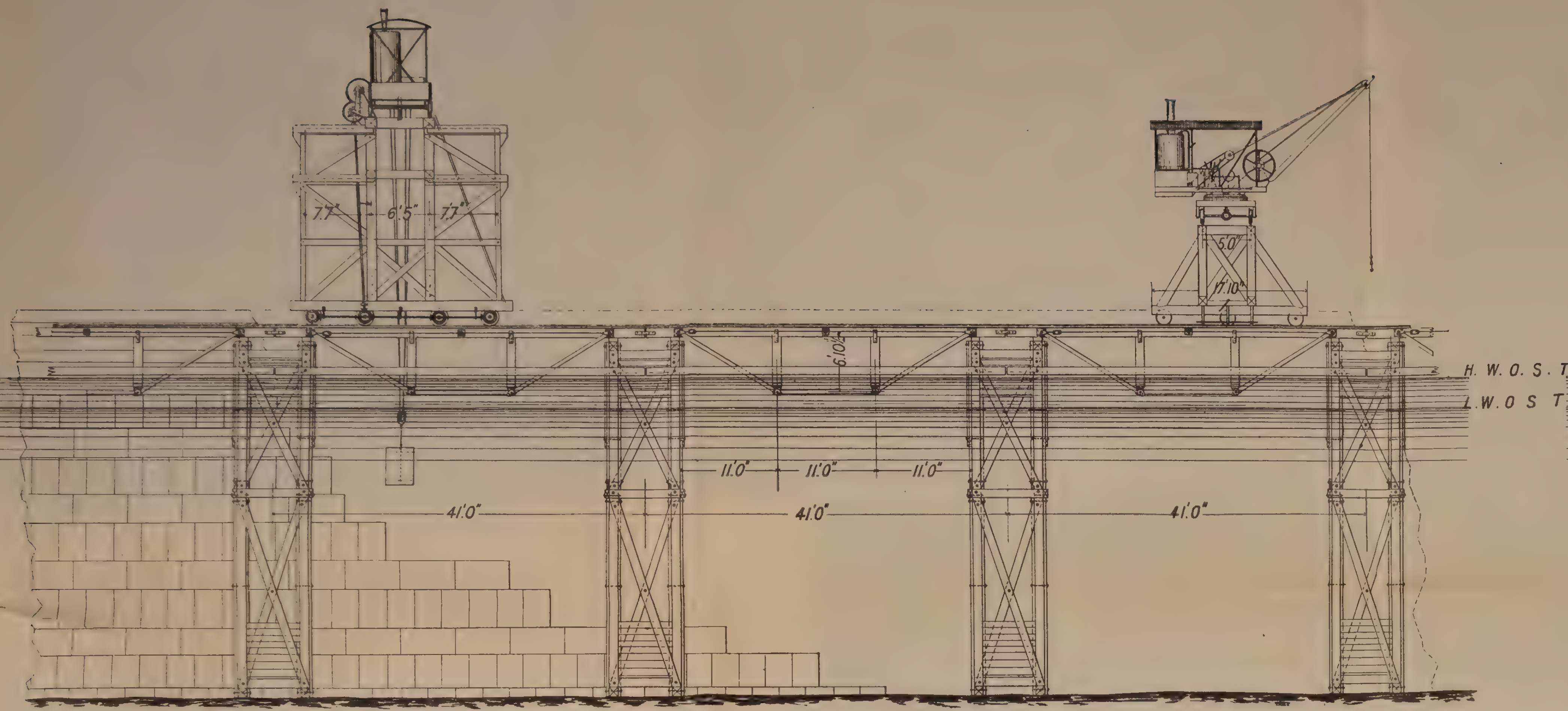
Staging for wall construction has necessarily to be very strong and substantial, in order to carry heavy-block setting appliances and concrete-mixing machinery, and this causes it to be a somewhat costly method, so that it is rarely employed except where the work is of some considerable width, as in the case of an inner quay wall built in conjunction with the breakwater proper. At Gibraltar, stages were only used when the breakwater was intended to have a quay wall on one or both sides, and was either 60 or 100 feet wide (see figs. 154–159). The island breakwater, consisting of 30-ton blocks, laid on a rubble mound foundation, was built by Titan cranes in the manner next described.

The End-on System.—The low-level system practised with a single powerful crane running upon a track laid over the finished portion of the work is generally open to the objection, already stated, of limited scope. The work proceeds outward from the land, and it cannot be attacked from several points as in the case of staging.[1] Yet the method is one which has been adopted in a very great number of modern instances. A strong point in its favour, particularly when dealing with huge blocks of 30 to 50 tons and more, is the greater stability of the working base. On the other hand, there are many occasions when its full lifting power is not in request, and when a much less powerful machine could do the work required at the moment.[2]

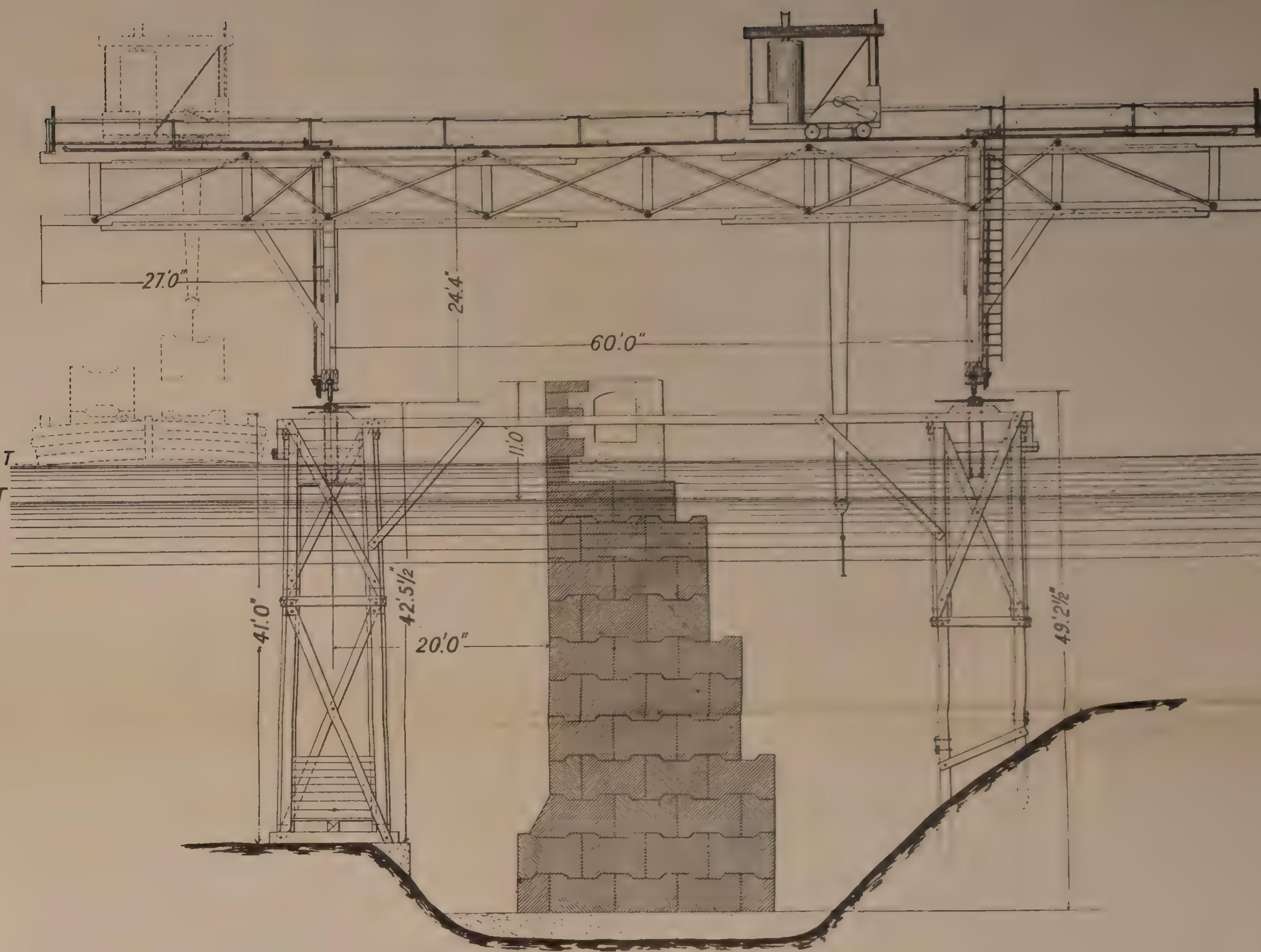
The machine employed in connection with this system of construction

[1] In the case of the Island Breakwater at Gibraltar two cranes were able to work simultaneously in opposite directions, starting from a central point, but this involved the prior construction of a base upon which to erect the cranes, and represents a special case which does not detract from the general consideration stated.

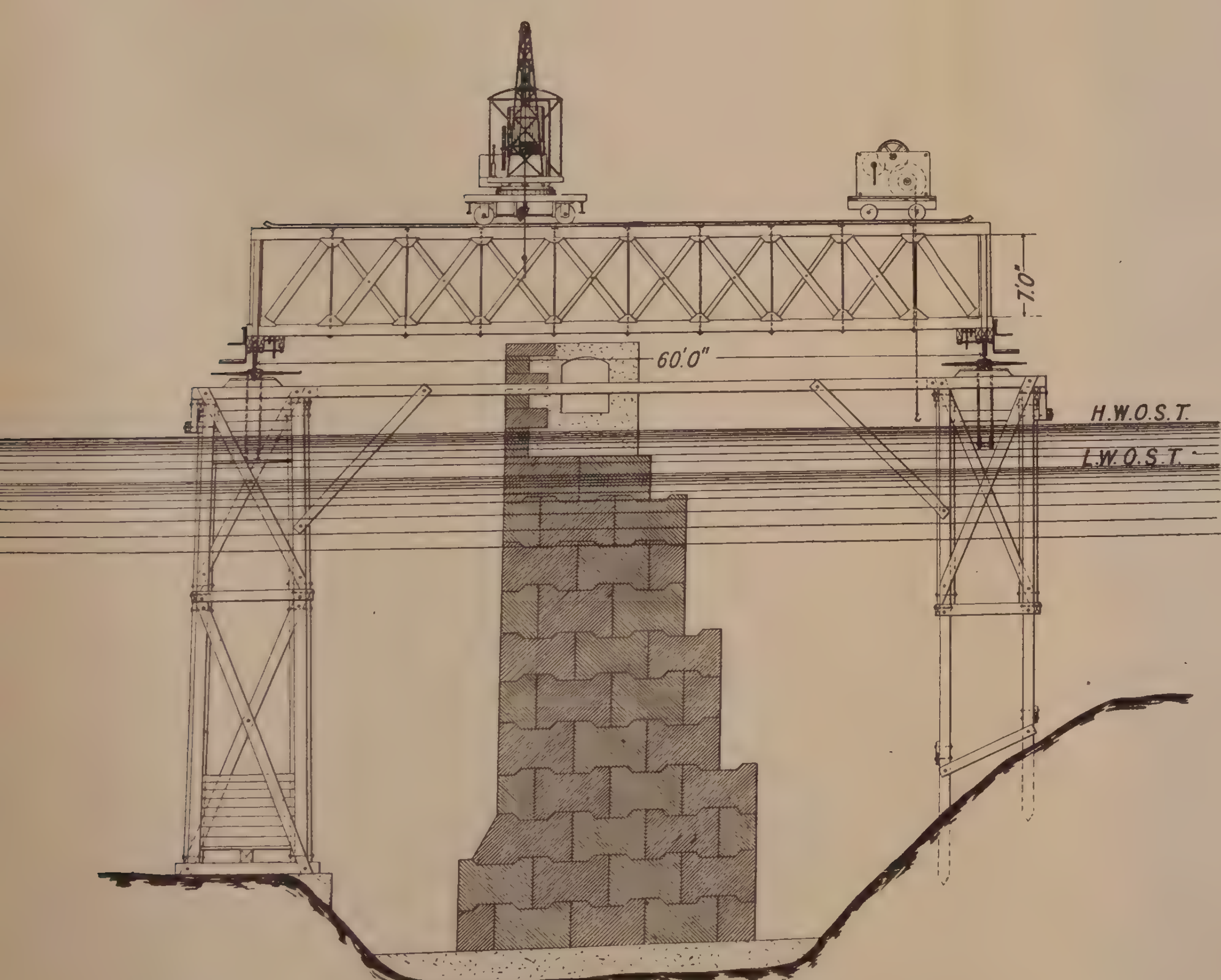
[2] Modern cranes are usually fitted with two sets of gearing: a quick set for handling light loads, in addition to the slower set for dealing with heavy lifts.



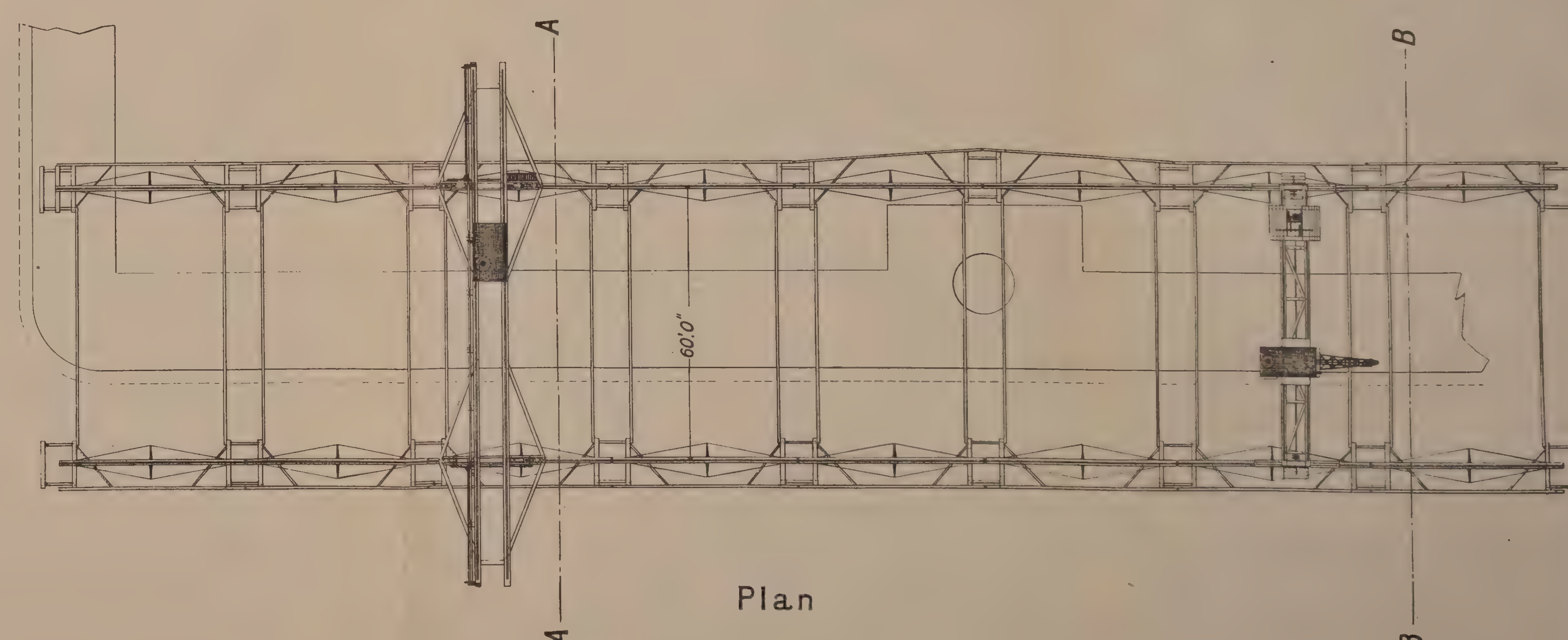
Elevation



Section on Line A.A.



Section on Line B.B.



Plan

Scale for Elevation and Sections, 16 feet = 1 inch.
,, Plan, 32 feet = 1 inch.

Figs. 154, 155, 156, and 157.—Constructional Staging at Gibraltar Harbour Works.

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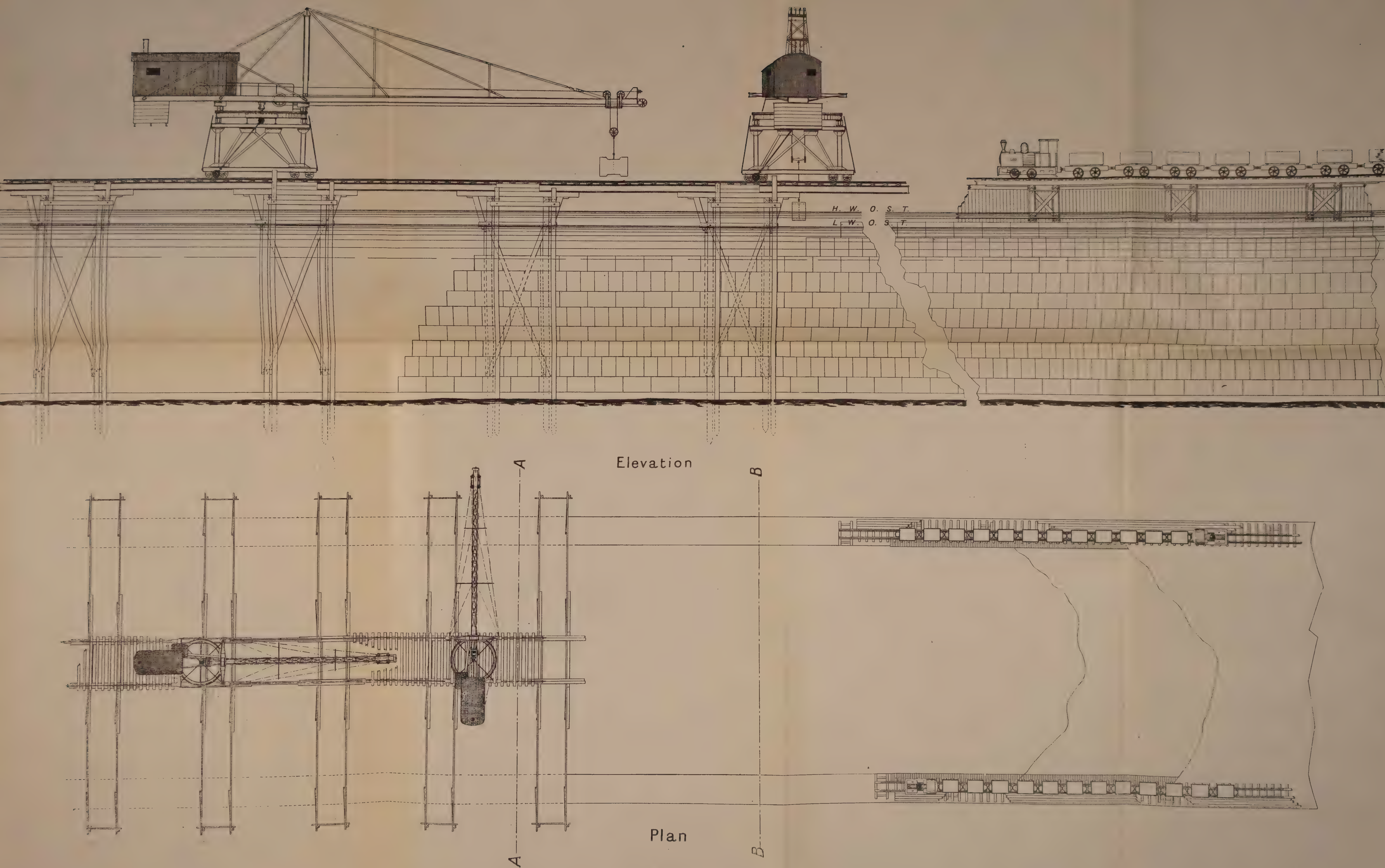


Scale for Eleavtion, 16 feet = 1 inch.
,, Plan, 32 feet = 1 inch.

Figs. 158 and 159.—Constructional Staging at Gibraltar Breakwater.

goes by the generic name of a "Titan" (fig. 160). In principle it consists of a huge cantilever crane with a substantial wheel base. There are two variants in design. In one case the cantilever arm is a girder trussed within its flanges; in the other it is supported by means of tension rods from above. The former obviously lends itself to greater stiffness and steadiness, while the latter is lighter and carries the arm at a lower level for the same over-all height.

Apart from their systems of trussing, Titans differ in that some have a fixed base, while others are pivoted upon their carriages. The former class, generally differentiated by the term "Mammoth," is provided with a carrier having longitudinal and transverse motions; the action of the latter class is radial. The radial machines can command a wider lateral range than the rectilinear machines, but they are not so conveniently adaptable to setting out work, a diagonal movement being less easily regulated to alignment in dual directions than a direct one. However, radial machines are capable of depositing wavebreakers along each flank of a breakwater to some distance outside, and this is a feature in which they decidedly excel the alternative type. Also,

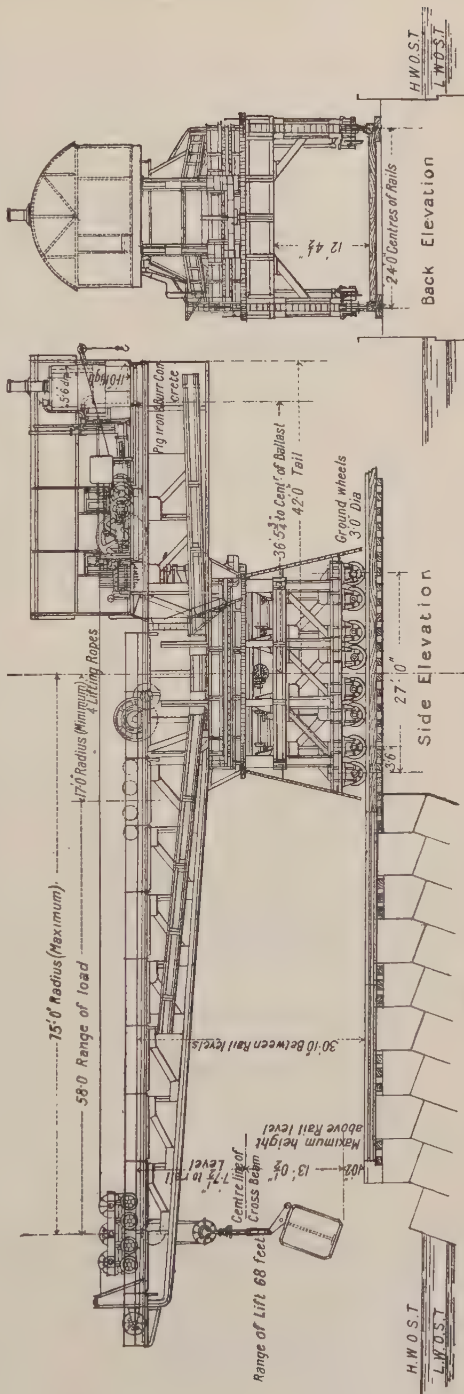


Fig. 160.—Titan.

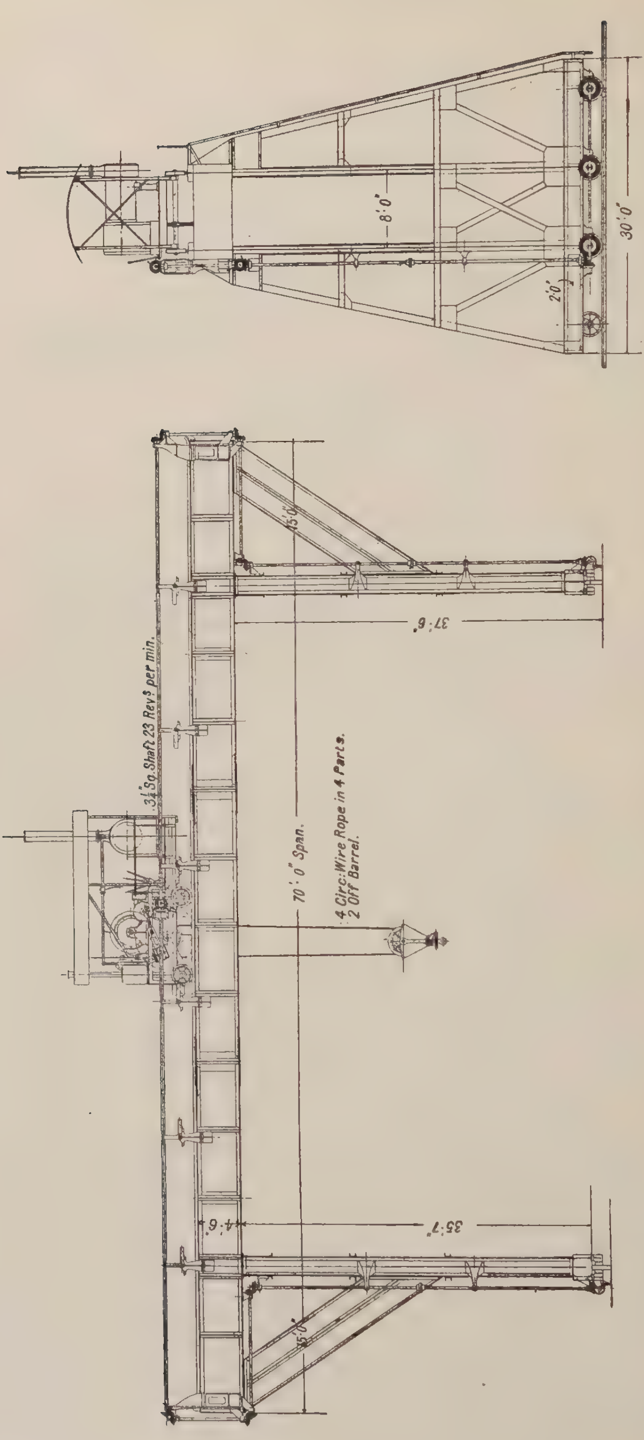


Fig. 161.—Goliath.

there is a considerable advantage attaching to a radial machine in building the roundheads or pier heads usually placed at the extremities of breakwaters. Moreover, with Mammoths, the block has to be run under the machine before it can be picked up, but with Titans this is not the case. This consideration is not unimportant, owing to the moorings.

The Titan is served with monoliths by a "Goliath" (fig. 161)—the generic name for an overhead traveller, the carrier of which runs on tracks transversely to the road of a wheel base of considerable span. The blocks are loaded on to trollies by the Goliath, and so conveyed from the block-yard to the breakwater, there to be set in position by the Titan. There is, however, nothing rigorous about the practice. The yard machine may be, and is, in some cases, a Titan.

Examples of both these machines are shown in the accompanying figures.

The **Caisson System** is an adaptation of the power of natural buoyancy to transportation purposes. Gigantic boxes of iron framework incased in concrete are formed in a sheltered recess or inlet on the coast or in an inner dock. When built to the required size—which is such that when sunk in position their topmost edges will project slightly above the surface of the sea at low water,—they are temporarily strutted in the interior, launched, and towed out to the site they are intended to occupy. Great care has to be exercised in aligning these huge boxes and in maintaining their perpendicularity while foundering. When this delicate operation has been successfully performed by admitting water to the interior of the caissons, they are filled with fluid concrete, stone rubble, and small blocks, so as to form ultimately a solid monolith.

The method involves some risk, especially on an exposed coast. The caissons are very unwieldy: they call for powerful towing and directing appliances; but once in position and rendered solid throughout, they offer a most powerful resistance to heavy seas. As regards cost, it is not apparent that they are more expensive than other forms of breakwater, but works on which they are adopted are liable to stoppages and delays arising from tempestuous weather.

Apart from and independent of any particular system of excavation, there are general features of breakwater construction which call for careful consideration.

The first and most important of these is the foundation.

Foundations.

It would be impossible almost to devote to this subject more attention than it merits. Very great and serious harm may accrue to a breakwater founded upon a base insufficiently firm and secure. Even if the damage be remediable, there is the expense of repairs, which will probably become a

matter of periodic recurrence. These repairs will naturally be of a more pronounced character in the case of regularly bonded structures, such as walls of ashlar work, which, when disturbed or deranged in any way, involve the provision of special appliances and skilled labour to reinstate them.

Accordingly, it will be well to consider the characteristics and qualifications of a good foundation. These may be classed under two heads: Incompressibility and Permanence.

Incompressibility.—A theoretically ideal foundation is incompressible: it does not yield in any way to the load imposed upon it. Such a foundation, however, except in the harder varieties of rock, is almost impossible of realisation.[1] The greater part of the material constituting the sea bottom is more or less of a compressible nature, though in some cases the compression may be but slight. Thus, in addition to the softer kinds of rock, sand and gravel and some varieties of marl are very little, if at all, affected by heavy loads, provided precautions be taken to prevent lateral escape. All other materials are compressible to a marked degree: mud, silt, the softer kinds of marl, clay (particularly when moist and plastic), peat, etc.

While an incompressible foundation is undoubtedly desirable, some slight yielding is no insuperable objection, provided the settlement be uniform. It is of no great moment if the whole superstructure sink a little; but if a portion only gives way, fracture between the stationary and yielding parts is bound to occur. Hence, a foundation should be as far as possible homogeneous. A building is safer on an all-clay foundation than on one of rock and clay. Where the foundation is varied in character, therefore, special precautions are necessary to ensure equal bearing power. The pressure on the weaker material should be distributed over a larger area; the dividing line between the two strata should be distinguished by augmented bond, such as is afforded by tie-rods or bars; and great care should be exercised in construction. The better course, wherever practicable, is to excavate to the lower level, at which the harder stratum is found.

It must be borne in mind that some settlement is inevitable. It will take place, if not in the foundation, at anyrate in the structure itself, especially in mounds formed of rubble work. The numerous vacuities in the mass and their proportionately great volume, combined with inequalities of bedding and support, lead to a shrinkage of the entire mass, which is very considerable in the earlier stages of its existence, and is more or less a constant characteristic. The diminution arising from this cause, however, is readily made good in ordinary cases by the simple deposition of additional material; but it is manifest that where the mound is acting as a substratum or base for a wall, the effects of shrinkage cannot be so easily effaced, nor can the wall itself escape a share in untoward consequences. Hence the obvious necessity of allowing such mounds adequate time to take a firm bearing.

[1] When obtained, it is not an unmixed blessing, as the levelling of an indurated surface is troublesome.

Moreover, it must not be overlooked that in addition to that arising from its own inherent tendencies, some further subsidence must occur when the weight of the wall is imposed upon a mound. Allowance must be made, in the first instance, for this and for other contingencies.

Settlement, therefore, in some form or other, must be looked upon as inevitable, and the essential point is to ensure its uniformity. Well-constructed breakwaters have sunk to the extent of 10 or 12 per cent. of their total height without appreciably affecting the appearance or the stability of the superstructure ; but this has only been so because the process was gradual and regular. Sudden and irregular changes cannot fail to produce fracture, especially in bonded work, concerning which we must speak later.

Permanence.—The second point of a good foundation is permanence, or unalterability. Certain mineral substances, when exposed to external influences, undergo physical and chemical changes which naturally modify their characteristics. The hardest rocks, such as granite, are known to disintegrate and decay under atmospheric agencies alone. Marine, and particularly submarine, agencies are much more drastic in action. The attacks of organisms, the erosive power of currents, the dissolving properties of water, and the percussive action of waves—all these are sources of change and deterioration.

As far as possible, therefore, a foundation should be guarded from destructive influences. Even when the ground is naturally firm and durable, it is very desirable to protect the surface in the immediate neighbourhood of the breakwater from scour. To this end, in the case of upright walls, rubble and riprap are deposited alongside, so as to form an apron covering the toe, and, in more exposed cases, large blocks and monoliths are similarly utilised.

Wall Foundations.—Before dismissing the subject of foundations, we must make a few remarks on the manner in which they are prepared for breakwater piers of the upright wall type.

In all cases, it is essential to remove the surface coating of mud, ooze, and weed, which covers the sea floor. This may be done by dredging or with the aid of divers.

If the stratum thus exposed be sufficiently firm for the purpose, the breakwater pier may be laid upon it forthwith. Otherwise it will be necessary to excavate further until a satisfactory base is obtained. In the event of the desirable stratum lying at a great depth, as revealed by borings, shafts may be sunk and the work built up in the form of piers inside them (fig. 162).

These shafts may consist of steel plating with walings of steel tees and angles, strutted with similar sections or with timber balks. They are usually so built, in lengths of convenient dimensions, such as can be handled by a crane or other lifting appliance. The lowermost length is fitted with a V-shaped cutting edge of hard steel. A sufficient number of lengths are bolted

together to bring the shaft above the water level when resting on the sea bottom. Excavation is then carried on in the interior by grab buckets with frequent inspection by divers. As the shaft sinks under its own weight, combined with that of kentledge, additional lengths are added at the top. When the solid stratum is reached, the interior of the shaft is filled with concrete. The spaces lying between successive piers are arched over at or about the level of the sea floor (fig. 162).

Another method of transmitting the weight of a breakwater to a lower stratum, is by means of concrete or timber piling driven at short intervals over the whole area of the site. The required depth must, of course, lie within the range of ordinary piles, say from 40 to 50 feet. Piles of greater length, if of timber, are expensive and difficult to obtain. When driven to their utmost extent, the heads of timber piles are cut off by divers and cased in rich concrete (say 3 to 1) to a depth of at least 2 feet below the mud level in order to secure immunity from vermicular attack. A foundation layer of concrete may then be distributed over the whole area.

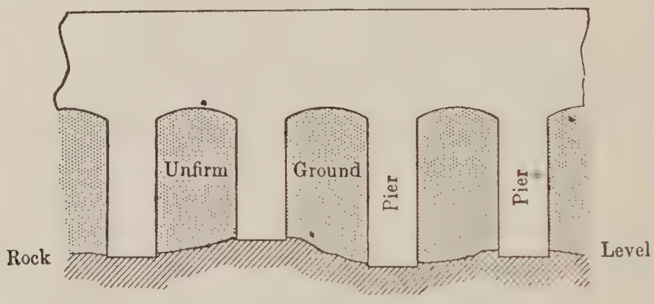


Fig. 162.—Pier Foundations.

In certain circumstances it may suffice to inclose the site within sheet-piling; remove an upper layer of material, a foot or two in thickness, and deposit concrete.

Limiting Loads.—In all these cases it is necessary to bear in mind the limiting resistance to compression of the stratum founded upon. The following values may be adopted for use in ordinary cases.

Concrete will safely stand from 10 to 15 tons' compression per square foot of area; hard rock from 9 to 10 tons; soft rock and stiff clay from 2 to 3 tons; and sand and gravel from $1\frac{1}{2}$ to 2 tons, if unconfined; if prevented from escaping laterally, the pressure may be increased to 3 or 4 tons.

Timber piles driven to a hard bottom will support a load of 10 cwts. per square inch of cross-sectional area; if dependent entirely upon the frictional resistance of the ground against its sides, and not upon basal support, the bearing power will vary with the perimeter of the pile; but in any case not more than 2 cwts. per square inch of sectional area should be imposed.

The loads actually due to the substance of a breakwater may be computed from the following table, which gives the weight in lbs. per cubic foot of

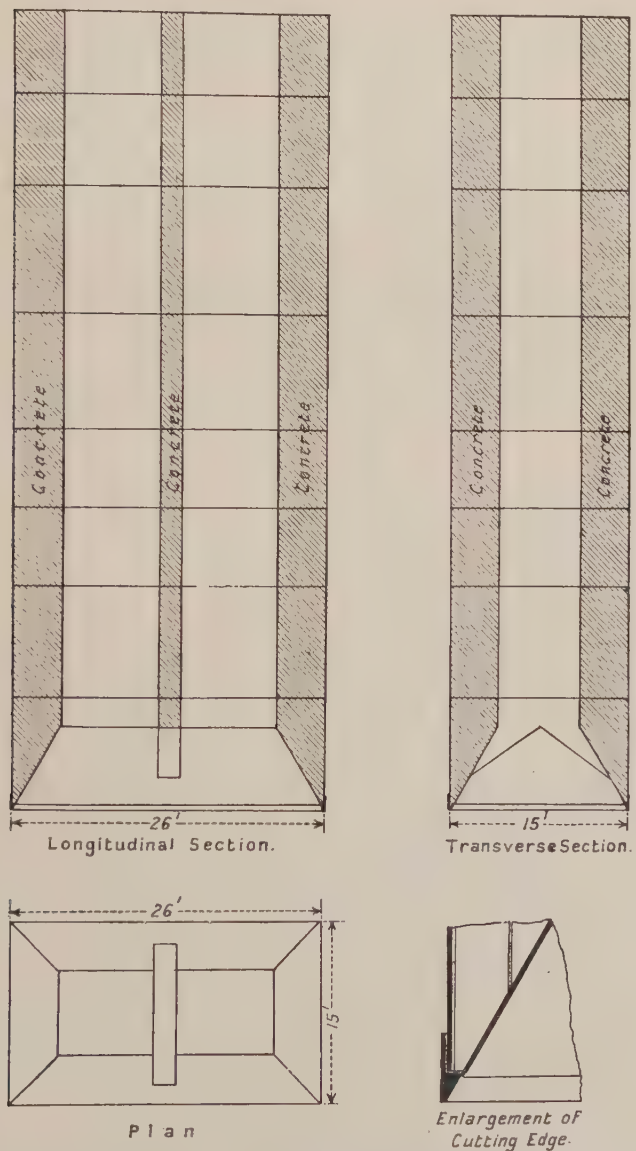


Fig. 163.—Foundation Caisson for Piers.

various minerals. When completely immersed in salt water, they lose 64 lbs. of the weight given; but there are circumstances under which the deduction is not justifiable, at anyrate for purposes of calculation.

Approximate Weight per Cubic Foot of Mineral Substances.

	lbs.		lbs.		lbs.
Basalt,	187	Limestone—*contd.*		Sandstone—*contd.*	
Brick,	115 to 135	Purbeck,	150	Talacre,	150
Granite—		Chilmark,	155	York,	157
Cornish,	164	Kentish rag,	166	Dundee,	159
Aberdeen,	166	Marble,	170	Monmouth,	168
Guernsey,	187	Magnesian,	175	Slate—	
Limestone—		Masonry,	116 to 144	Cornwall,	157
Bath,	120	Sandstone—		Westmoreland,	173
Portland,	130	Red,	130	Welsh,	180
Chalk,	145	Craigleith,	141	Trap rock,	170

Surface Treatment.—The surface of a reliable natural foundation generally requires some treatment before it is ready to receive the first course of wall structure.

In rock there are always numerous cracks, crevices, and fissures, and a general unevenness of surface. Cavities and pockets containing soft material should be cleaned out and filled with concrete prior to extending the concrete over the entire site. To prevent lateral escape of the concrete while soft, it should be flanked on each side, temporarily, with bags of sand, planking on edge secured to iron pins driven into the rock, or by slabs of stone. Small irregular apertures may be staunched by packing with clay, or by covering with strips of jute or canvas.

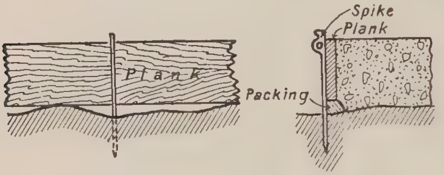


Fig. 164.—Moulds for Concrete Foundation.

At Famagusta, Cyprus, after a section of the sandstone rock had been prepared by divers, longitudinal planks were set on edge at a distance apart

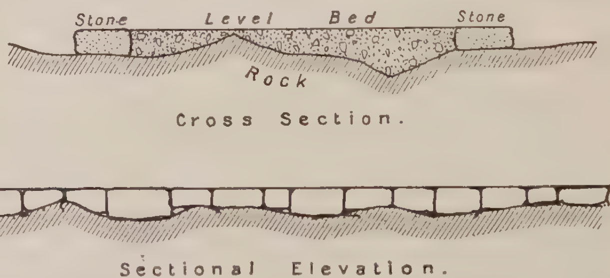


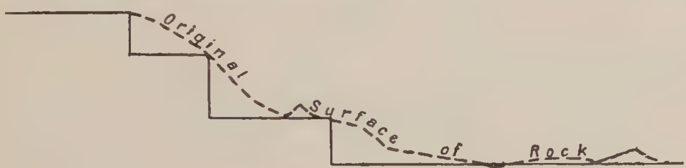
Fig. 165.—Concrete and Stone Foundation Work.

equal to the width of the base (17 feet) of the wall. The bottom edges of these were either scribed to suit the irregularities of the rock, or, in some cases, packed with concrete bags on the outside of the planks. The top edges were set to the exact level at which the blockwork was to commence. The planks, or shutters, were held in position by bolts fixed in the rock, and were secured

thereto by coach screws or spikes. Concrete was deposited within the space enclosed by the planks from a tripping bag, and a couple of divers, one on each side of the trench, levelled the upper surface by means of a light rail used as a straight-edge, after which care was taken that the concrete remained undisturbed. It was allowed to set for three days before any blocks were placed upon it.[1]

Where the top of the rock, however, is not very hard, it may be found preferable to dress it down to a level surface, or to a series of benched beds of sufficient area to receive one or more blocks. Dips should likewise, where possible, be benched out to prevent any tendency of the wall to slide over the sloping surface (fig. 166).

The levelling of the surface is, of course, only absolutely essential for blockwork. For walls built of concrete in mass, though benching is desirable, all that is strictly needful is to erect the side-moulds within which the concrete is to be deposited. At first sight this would appear to be a simple operation, but the difficulties of setting temporary wooden moulds under water are



Benching

Fig. 166.

anything but negligible. Where piling is practicable, a series of uprights may be driven at regular intervals, fitted with grooves within which panels of sheeting may be slid down, and raised again as the work proceeds. At the junction of the planking with the ground, a broad strip of canvas can be laid, forming a lining to the adjacent surfaces of each, for a width of 18 inches or 2 feet. The vertical portion will be backed to the planking, and the horizontal portion weighted to the ground with stone.

For mass concrete on a rocky bed, where guide piles are impracticable, the plan to adopt would be to lay external facing blocks and to deposit mass concrete in the space inclosed.

The deposition of concrete under water is an operation requiring the utmost care for its satisfactory accomplishment, the danger being that the cement may be washed out of the aggregate. It is useless, therefore, to entertain the idea of tipping, as carried out in ordinary work above water. For the special circumstances of subaqueous foundations, the concrete must be conveyed in a skip with a bottom flap or flaps, or in a bag with a double mouth, that at the lower end being temporarily bound with a looped rope, capable of being released by a tripping rope. The skip, or bag, is lowered

[1] Hobbs on Famagusta Harbour, Cyprus, *Min. Proc. Inst. C.E.*, vol. clxxvi.

right to the bottom, or as near thereto as is consistent with discharge, and the contents are allowed to flow quietly into place, with as little manipulation as possible. It will be evident that concrete for submarine work should be rich in cement, say 4 to 1.

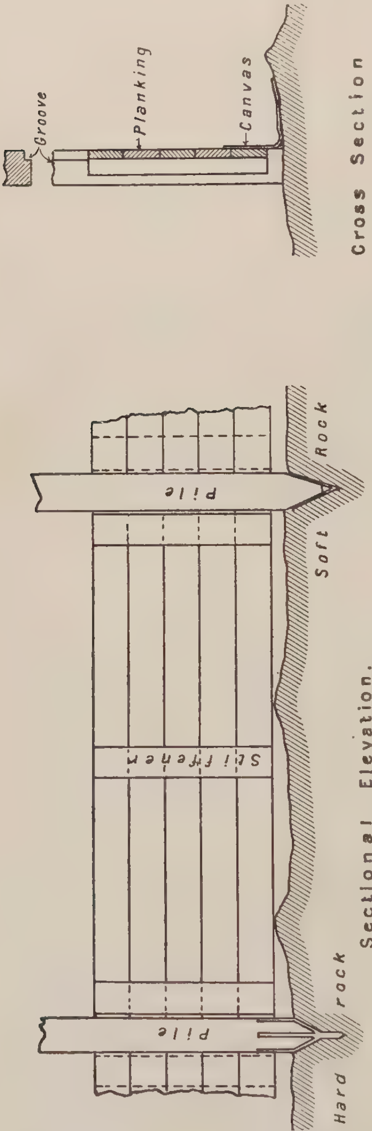


Fig. 167.—Moulds for Reception of Concrete under Water.

One caution to observe in tidal situations is that of so arranging the periods of deposit that a mass of freshly placed concrete may not be subjected, while setting, to the disturbing action of a choppy sea surface. Less than 18 inches or 2 feet of water is insufficient to prevent even small waves from exercising a deleterious influence, chafing the concrete, and robbing it of its cement. Therefore, wherever possible, advantage should be taken of the variation in the tidal level, during springs and neaps, to suspend concreting from time to time, either at some little depth below the surface or altogether out of range. A quick-setting cement will prove of considerable value in tidal situations.

It is needless to remark that when operations are carried on within the shelter of a diving-bell, the same restrictions do not apply, although it must be admitted that sudden outbursts of air may do more damage than the fretting action of waves. Yet, with care, these may be avoided. The work is then only limited by the convenience of arrangements in regard to shifts.

Bagwork.—The dispersive action of waves, and, indeed, the solvent action of comparatively still water, has led to the introduction of a system of concrete laying in sacks or bags. These bags have, in certain cases, attained a very considerable size and weight, the latter reaching 100 tons and over. But small bags of 5 or 10 tons, or thereabouts, are most common. They are often employed for regularising the surface of an uneven bed destined to receive blocks. The

bags are of jute or canvas, strongly made. After being filled with concrete, they must be deposited immediately, while the material is plastic, so that each bag may adapt itself to the inequalities of its environment. This adaptation of bulk to various positions is one of the chief advantages claimed for bagwork. The system is not without certain drawbacks; the bags are liable to burst. This, of course, could be remedied by strengthening and improving the sacking. Moreover, the bags cannot be brought to a perfectly level surface; neither can they be compacted very closely in successive rows; and further, they are liable to work loose and be sucked out by the sea, or, failing that, the ends may be broken off by waves. These defects, however, are not vital; careful setting will go far to minimise them; and many breakwaters in existence have been partially, or wholly, constructed of bagwork.

The jute sacking generally used for the purpose weighs from 25 to 30 ozs. per superficial yard.

Block-making.—The making of blocks for breakwater building calls for little explanation. The blocks are built of concrete in moulds at a block-yard adjacent to the site. The weights range from 5 tons upwards, according to the capacity of the setting machine. There is, however, practically no limit to size, since huge monoliths may be deposited by special means. At Dublin, as already stated, a wall has been built with foundation blocks weighing 350 tons each, while the caisson blocks at Zeebrugge weighed no less than 4,500 tons each.

Blocks should be allowed to mature, if possible, for a couple of months—one month, at least—before depositing in position, though they may be removed from the moulds at the end of a fortnight. The season of the year and the temperature produce variations in the time of maturing.

In order to facilitate the placing of blocks, they are usually constructed with two vertical or slightly inclined perforations, through which are passed iron bars with **T** or angle ends, capable, when turned through a right angle, of engaging in recesses formed in the underside of the block. When the blocks are very heavy, these recesses should be provided with hard wood or iron bearing surfaces to prevent the concrete from suffering damage.

The following details of the block moulds used in connection with the building of the Tyne North Pier will serve to illustrate some of the points to be considered and the precautions to be adopted[1]:—

"The sides of the block-moulds were formed of horizontal planking, 3 inches in thickness, each side being tied together by three vertical external battens and each end by two. The ends were both the exact width of the block to be moulded, but the side flitches were made to overlap them to the extent of 1 foot at each end. Two vertical battens, 5 inches by 2 inches in section, were firmly attached to each of the side flitches near the respective ends, and these were placed at such distance apart that when the end flitches

[1] Barling on Tyne North Pier, *Min. Proc. Inst. C.E.*, vol. clxxx.

rested against them the enclosed space was of the correct length for the block that was to be made. The side flitches were held together, and were make to grip the end flitches by means of three horizontal straps with screwed ends, which latter passed through slotted holes in the side flitches, tailed nuts being fitted to the screwed ends. Before a mould was filled it was strutted externally near the bottom, and the sides were further stayed by means of two $\frac{7}{8}$-inch bolts passing through vertical battens at the centre, one bolt through the centre of the block, and the other above the mould, the batten in question being prolonged about 6 inches for the purpose. The top edge of each mould was protected by a plate of iron 3 inches by $\frac{5}{16}$ inch, and a vertical angle bar, 4 inches by 3 inches by $\frac{3}{8}$ inch, extended from top to bottom of the mould at each point where the bolt-ends or 'lamb's tails,' as the men called them, passed through the side flitches. The foregoing construction enabled the mould to be separated from the block to the extent of about 2 inches without it being necessary to take it apart. To facilitate quick stripping, thick slotted washers were used between the tailed nuts and the side flitches; these washers were removed as soon as the tailed nut was slacked, and thus the slacking back of the nuts was reduced to a minimum. Near the centre of each end flitch, two vertical horns projected about 6 inches above the top of the mould to form a support for a deal extending from end to end of the mould. To this deal were fixed the two tapered hardwood cores which formed the holes in the block for the lifting **T**-bars. Great care was necessary in strutting the moulds and in tightening the through bolts near the centre, as otherwise the blocks would have been considerably wider in the middle than at the ends, a defect which would have increased as the mould became older and been fatal to close joints." Considering the trouble which this involved, the Resident Engineer was of opinion that the side flitches of, at any rate, the longer moulds should have been 4 inches thick instead of 3 inches.

"Before filling the mould, the floor, itself of concrete and perfectly smooth, was sprinkled with fine sand, in order to prevent the concrete from adhering to it, and for the same reason a specially prepared emulsion [1] was applied to the sides of the mould and to the core bars; soap and water was at first tried for the latter purpose, but it was not entirely satisfactory. The blocks in all cases were 6 feet in width, but both height and length varied, the contents ranging from $5\frac{1}{3}$ cubic yards for the smallest to just over 19 cubic yards for the largest. All exposed blocks were faced with granite, grey Aberdeen being used exclusively above water and pink or grey being allowed indiscriminately below When a face-block was being moulded, the facing took the place of one of the end flitches of the mould, but the same care was required in attaching the built face to the side flitches as when hearting-blocks were

[1] The efficiency of the composition was undoubted, but all the author knew of the composition was that it was an emulsion; it was sold as "concrete oil," and from its comparatively low cost he suggested that its constituents were of the nature of by-products. *Ibid.*, p. 213.

being moulded and a wooden end flitch was used. In each face block, cavities, semi-circular in plan, were moulded, a single pair in the short face-blocks and a double pair in the long ones; the hearting-block immediately behind the former was also provided with a single pair, which, when the blocks were set, came opposite the inner pair of cavities in the long face-blocks, the outer cavities corresponding in a similar manner with those in the short face-blocks. On the blocks being set in place the cavities were filled with 4 to 1 concrete, cylindrical bags being used to contain this when employed below water. The joggles thus formed act as keys, which effectually prevent any face-block from being drawn out.

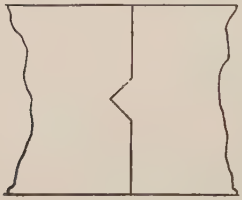
"The lewis cores were drawn and the sides of the moulds were stripped at the end of 24 hours after filling, but the blocks were allowed to remain undisturbed on the floor, where they had been moulded, for a week, when they were lifted and stacked. A pair of the ordinary form of **T**-bar lewises was used for lifting, but many of the blocks were so heavy that the greenheart bearing-pieces, moulded in the blocks for the **T**-heads to rest upon while lifting, were not hard enough to withstand the severe stress, and it was eventually found necessary to plate them with ¼-inch iron."

Bond.—The problem of bond in breakwater construction is a difficult one. Theoretically, the effect of introducing a system of interlocking is to strengthen materially the breakwater by binding together in close association the separate elements of which it is composed. Practically, there are the consequences of unequal settlement to be considered, whereby the sinking of any portion of the breakwater will probably entail fracture between that and the part adjacent. Particularly is this the case in composite breakwaters where a coursed wall rests upon a rubble foundation, overlying, in turn, a natural bed, which is most probably of a compressible nature. The evils attaching to the liability to irregular settlement can only be averted by discarding the idea of bonding horizontally. Vertical or sloping joints then become inevitable. The breakwater can still be connected longitudinally by means of dowel or joggle joints, which offer little or no resistance to settlement.

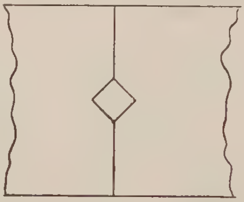
A *dowel* joint consists of a square-shaped or circular aperture, set diagonally, one-half—*i.e.*, a **V**-shaped or semi-circular portion being cut out of each of two stones. When these are joined together, the aperture is filled with a piece of stone of the required section, or with concrete. A *joggle* joint differs only from a dowel joint in that the connection is formed by a projection on one piece fitting into a recess in the other. It is the stronger joint of the two, but more expensive, because a considerable quantity of one stone has to be cut away in order to form the joggle.[1] In the case of concrete blocks, this objection, of course, does not apply, as the projection is moulded when the block is made.

[1] The distinction in nomenclature between a dowel and a joggle is, however, not always observed, and the terms are often used indiscriminately for either type of joint.

Where walls are founded directly on a firm, homogeneous stratum, such as hard shale or rock, the horizontal bonding of the blocks is quite permissible, and has been practised with satisfactory results. At Dover, for instance, the blockwork rests immediately upon the chalk, and there is no record of fracture in the bonding. The pressure on the foundation did not exceed 4 tons per square foot, and this restriction of the load within the limits of the compressive resistance of the material is, of course, an essential condition. At Gibraltar a system of interlocking was adopted for the North Mole, where the foundation is hard shale and conglomerate, the surface of which was levelled to receive the blocks with a thin layer of concrete.[1] Even if there

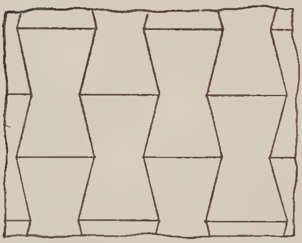


JOGGLE JOINT

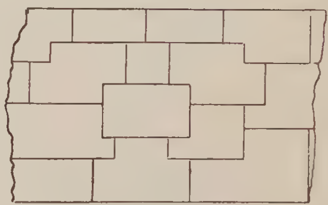


DOWEL JOINT

METAL CRAMP



DOVETAIL MASONRY (PLAN.)



PIANO-BLOCK MASONRY (ELEVATION.)

Figs. 168 to 172.—Masonry Joints and Connections.

is some slight settlement in the foundation, little harm is done, provided it is uniform and regular.

There are several methods of arranging horizontal bond. Plain, rectangular blocks may be used with variations in length, as at Dover, or the blocks may be moulded to special forms with corresponding projections and recesses, as at Gibraltar. Dovetailing (fig. 171) is another method, but it is unduly elaborate and very costly, and, therefore, not much used except for parts where great strength is required, as pierheads and lighthouses.

Somewhat similar, though perhaps less intricate, is the system of "*piano*" *blocks* introduced by Mr. Messent in the construction of the old Tyne breakwaters. The idea of using these deeply rebated blocks, shaped like piano keys (figs. 172, 173), was to prevent the sliding of the blocks transversely

[1] The coursed work in the South Mole and at the heads of the detached Mole is open to the objection that there is an underlying bed of rubble, which is of considerable depth.

when struck broadside by a heavy sea. The system, however, did not prove a success, and is unlikely to be repeated.[1]

The object aimed at has been achieved, in the case of the new North Pier (fig. 145), in a much simpler manner by the provision of a check about 6 inches deep extending the full length of the breakwater.

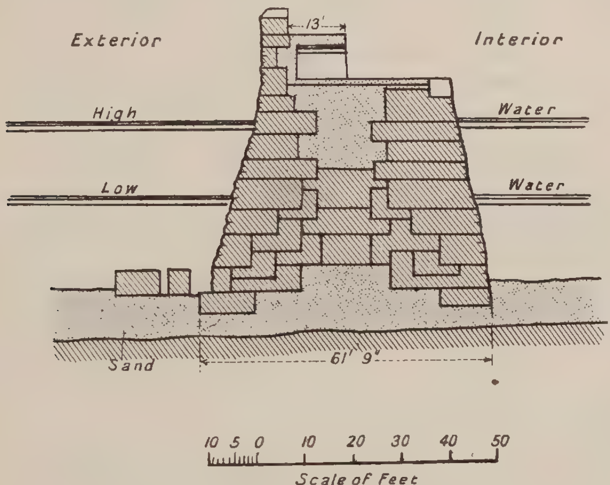


Fig. 173.—Piano-blockwork at Tynemouth.

An alternate method to prevent horizontal movement is by means of *bed-plugs*. These are projections of stone or iron standing up above the level of one course of blocks and fitting into apertures in the underside of the next course.

Metal cramps form an effective connection, provided they are protected from the possibility of rusting and corrosion. This can only be satisfactorily realised by bedding them below the surface of the stone and completely inclosing them in Portland cement.

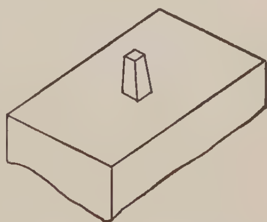


Fig. 174.—Bed-plug.

Sloping Bond or **Slice Work.**—The term sloping bond or slice work has been applied to an arrangement of blocks whereby they lie tilted a little out of the vertical—the angle of inclination varying generally from 60 to 75 degrees. By this system the blocks are fairly free to slide, in case of settlement, without disturbing the adjoining courses. When, however, as in some cases, dowelling and bed-plugs are also introduced, this freedom of action exists onlyto a restricted extent; the frictional resistance to movement is itself considerable.

[1] That is, on any extensive scale. It has been used in situations where the special labour and cost were justified by the circumstances. Thus, at the junction of the breakwater extension at Madras with the old work, an abutment for the sloping blocks was formed by the introduction of some piano blocks into the vertical coursed work.

The horizontal bonding of blocks—of dubious advantage as it is in the upper part of a breakwater where the blocks can be accurately adjusted, and the bed-joints well flushed with cement—is a matter of almost positive harm in the courses which lie below water level, where, in most cases, blocks have to be laid without bedding, and where the joints are left open. It is manifest that, under such circumstances, the blocks are not bearing equally on their beds, and it is readily conceivable that a long block extending over three others in a lower course might only be supported at each end. The risk of fracture would then be very great.

Fig. 175.—Angles of Sloping Bond of Breakwaters. See also fig. 160, p. 217.

During the progress of the work, and especially at the commencement of a winter season, or other period when operations are intermittent or entirely suspended, care should be taken to see that the end blocks of the work actually executed are amply secured.

The use of sloping bond commonly involves some special arrangement in regard to the moulding and setting of the blocks. In the majority of cases[1] the bottom course is laid upon a horizontal bed, and moulds of corresponding form are consequently required for the blocks in this course. For the other courses the ordinary rectangular moulds suffice, but the blocks have to be tilted to the required angle when lifted for setting. In practice it is found convenient to mould the blocks in a vertical position—that is, in the ordinary way for horizontal blockwork. This generally necessitates a second set of lewis holes to tilt the block to the required angle, or else some form of tilting gear. Of this latter arrangement, the Fidler device is a suitable example. The lewis bar holes are formed in the usual positions perpendicular to the block-making floor, and the bars are lowered into them. Shortly before the bars reach the bottom of the holes, rollers in the steel crosshead take a bearing on the upper surface of the block and roll across. In so doing they alter the relative positions of the lewis bar and the lifting bar, so that they are no longer in a vertical line, and when the block is raised it inclines to the desired angle.

That special lifting arrangements are not absolutely essential is evident from the experience at Madras Harbour Remodelling, described in the following extract from Mr. Mitchell's paper[2]:—

[1] As at Colombo, Madras, Manora, and elsewhere; but, at Gibraltar, blocks of rectangular form were employed throughout and the underside of the bottom course was packed to a saw-tooth profile by means of triangular-shaped bags of concrete.

[2] Mitchell on the Alteration of the Form of Madras Harbour, *Min. Proc. Inst. C.E.*, vol. cxc.

"The slice-blocks were in three sizes, 14 feet, 12 feet, and 10 feet long, and were all 6 feet wide by 5 feet 9 inches high. The beds of the blocks when set in the works were at a slope of 1 to 4 with the horizontal; but the blocks were made on the flat, the cores for the lewis bars being set at the required angle. No difficulty was experienced in lifting these blocks after ten days with the ordinary lewis bars. They canted to the angle of the bars while being lifted, without bending the bars or doing any damage to themselves. The possibility of thus canting the blocks, while lifting them from the horizontal bed on which they had been made, was largely due to the use of hydraulic Goliaths, which gave a very steady lift."

Grouting under Water.—The joints of work under water may be filled, to a certain extent, by means of grouting from the surface. A pipe or tube is arranged so as to communicate with the part proposed to be dealt with, and through this tube, under a considerable head or under direct pressure from a ram or piston, fluid concrete is forced into all the adjacent cavities. To prevent the escape of the concrete, however, all the face joints must necessarily be closely caulked. This has been done in certain cases by forming slightly dovetailed grooves near the outer edges of the blocks and ramming them with rolls of canvas containing neat cement, as shown in sketch (fig. 176). These were allowed to harden, and the joints packed with shingle before grouting.

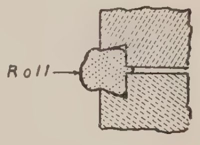


Fig. 176.—Joint Packing.

In the case of wide joints, the apertures may be faced with brickwork in cement, or with small bags, containing neat cement, stacked compactly.

The concrete for grouting purposes should not be too fluid. Other materials are also used, such as clay worked up with hydraulic lime, and sand mixed with iron filings and sal ammoniac; but Portland cement concrete, on the whole, is preferable.

Minor Breakwaters.

All breakwaters are not planned on the same scale. Massive construction may be both necessary and possible in the case of leading harbours and ports. But there are also small harbours, where any great outlay on protection works is out of the question, and where some expedient must be contrived for affording reasonable protection at moderate cost. It is both interesting and instructive, therefore, to consider the steps which have been and may be taken to meet these cases.

Crib and Box Breakwaters.—The submersible caisson of steel and concrete has had its prototype in a long, wooden box, floated out into position and filled with rubble. Such was the form of breakwater adopted in many

early instances, and still practised at some ports on the Baltic seaboard These boxes or cribs were braced, at intervals of 8 to 10 feet, by transverse partitions, and, in wide boxes, there were often one or two longitudinal partitions as well. Floors of planking were arranged in several—about three —tiers, with a charge of stone incased in each. Both the outer casing and the inner partitions were constructed of solid timber.

"Stone cribs of this type of construction constantly required repairs, since, apart from scouring, the terminal boxes were damaged by every strong sea; the planks and the upper balks were torn off, although secured by means of strong iron bolts, and the stones were hurled about." [1]

The dams referred to were strengthened, as far as possible, by driving piles through the inclosures, in two rows with cross ties, and by depositing a mound of huge stones in front of the seaward face. But no measures proved completely satisfactory. Breakwaters such as these could only be employed in comparatively shallow and but moderately exposed positions. Depths of 15 feet of water probably mark the limit to which they may be advantageously applied.

Somewhat similar to the foregoing are the timber cribs used on the North American Lakes. They are box-shaped frames of timber constructed in open work, with numerous compartments formed by means of transverse and longitudinal ties. The compartments form receptacles for stone rubble. From the crudeness of their build, these cribs can only be looked upon as of the nature of temporary structures. They are referred to in somewhat greater detail in *Dock Engineering*.[2]

Fascine Work.—Another form of construction adopted in certain localities for moles and breakwaters is known as fascine work, and consists of bundles of brushwood arranged as mattresses, which are sunk in position in successive layers and weighted with stone. Piles are then driven through the mattresses into the sandy bottom to prevent displacement. In process of time the interstices of the mattresses become filled with sand and drift, forming a solid mass. The system is more particularly characteristic of the low-lying coasts of Holland and Denmark, though it is also to be found on Prussian shores. Fascine mattresses are also described at some length in *Dock Engineering*,[3] and they are alluded to in Chapter X. of the present volume in connection with channel training works; but, as in the previous instances, they have exhibited no great resisting powers to rough seas.

The following is a description of some early fascine breakwaters on the Baltic littoral:—

"The fascine dams consisted, according to the depths of water, of one or several layers of fascines 3 feet to 4 feet thick, which were floated down from the inner harbour where they had been made, and sunk on the spot. The upper layer was afterwards covered with a packing and with a stratum of

[1] Anderson on Breakwaters, *Proc. Int. Nav. Cong., Milan*, 1905.

[2] 2nd edition, p. 286.

[3] 2nd edition, pp. 282, *et seq.*

small stones and rubble, about 3 feet in height and rounded on the top. This cover was paved with large, approximately cubical, stones of granite. The capping, which had a width of about 13 feet and was slightly arched, was scarcely 6 feet above mean water level. The slope from the capping down to the outer edge was 3 to 1 on the sea side and 2 to 1 on the harbour side. The thickness of the stone paving was 3 feet in the capping and 2 feet elsewhere. The stone layer was further secured by strong oak piles from 7 to 10 feet long and 6 inches square, called caisson piles; they were driven at distances apart of 6 feet along the edge of the capping, and of 18 inches along the water-line.

"These fascines were exposed to heavy damage, for every storm from the sea lifted the paving stones of the slope, especially at the head and on the sea side, from their seats, and carried them inland, or hurled them up the slope and over the mole into the inner harbour. The reason for this was mainly that the stones did not lie sufficiently close upon the flat slopes, and that they could be loosened separately and disturbed by the waves, lacking sufficient weight in themselves to resist this action. In order to make the surface of the slope as plane as possible, with a view to avoiding points of attack for the impinging waves, the stones had been placed with their roughly hewn, approximately square, heavy portions—that is to say, with their bases—upwards, and with the tapering parts downwards. In this position they were only secured by being packed underneath with smaller stones and by leaning against one another. The stones were not further fixed, therefore, and although they were as closely packed on the surface as was possible, the remaining gaps, especially between the lower portions of the stones, afforded the waves sufficient front for attack. It was for this reason that the joints of the stones were at a later period closed with concrete, when a quiet sea and low water permitted such operations. These measures diminished the destruction, but did not by any means prevent it; for the chief trouble arose from the insufficient loading of the stones, which could not be altered, and the part of the structure which was most exposed—*i.e.*, the toe on the sea side—could not be strengthened and properly secured. The toe of the slope on the harbour side could be strengthened by forcing in several layers of balks behind the pile walls, and it was against these balks that the cubical stones of the slope were resting. Nothing of the kind, however, was possible on the open water side. Rubble mounds were useless; for the stones were driven inland or raised up the flat slopes of the moles into the harbour. There was nothing left, therefore, but to place further fascines in front of the slope to fill up the hollows formed alongside the mole, and to restrict further injury to the pavement."[1]

Another simple method of breakwater formation is to drive a double row of piles and fill the intermediate space with rubble, the piles being retained in position by longitudinal walings and transverse ties. The piles may be

[1] Anderson on Breakwaters, *Proc. Int. Nav. Cong.*, *Milan*, 1905.

either of whole timber or of iron. In one case, at Touapsé on the Black Sea, railway metals were used for the purpose. On account of the disruptive tendency of the hearting, it is necessary to have good stout piling with strong transverse pieces capable of offering ample resistance to the lateral pressure imposed upon them. The piles are sometimes driven in an inclined direction, pointing inwards from the bottom towards the top so as to increase their stability.

Examples of Breakwater Construction.

North Pier at Tynemouth.[1]—The new North Pier at Tynemouth, replacing the old pier referred to on p. 205, consists of blockwork throughout. The blocks, ranging, as already stated, from $5\frac{1}{3}$ cubic yards for the smallest to just over 19 cubic yards for the largest, were prepared in a yard, distant over half-a-mile from the site, by means of two mixers, of the Messent type, fixed on timber staging. The charge for each mixer was 1 cubic yard of ballast, from the upper reaches of the Tyne, with two measured bags of Portland cement, giving a mixture of 6 to 1 by measure. The mixers were rotated by gas engines, each 12 B.H.P., and as much as 206 cubic yards were turned out by a single mixer in ten hours. The concrete was conveyed to the moulds in side tipping waggons, propelled by hand, and running on a way of 2-foot gauge, the level of which was about 15 inches above the top of the average block mould. The block floor was capable of holding 75 moulds in five rows. Two Goliaths upon a single road of 60-foot gauge controlled the block-floor, and about half the stacking-ground, while a third machine was installed in another part of the yard. Blocks aggregating 125,018 cubic yards were made, and the maximum stock at any one time was 30,700 cubic yards, the full capacity of the storage space. The blocks were conveyed to the pier by locomotives.

The staging (fig. 176*a*) carrying the three Goliaths, each of 65 feet span, used for preparing the bottom and setting the blocks, was of very strong construction. The supports were trestles or dolphins, each composed of six 16-inch square Oregon pine piles on the north side and eight such piles on the south, or harbour, side. The larger trestles on the harbour side were required for supporting the service road over which the blocks were conveyed to the point of setting. The trestles were placed in some case 40 feet, and in others 48 feet apart longitudinally. The piles were driven down through the rubble mound foundation of the old pier and the underlying sand into the hard shale, the shoe penetrating from 6 to 18 inches. The braces coupling together the piles of each trestle were old double-headed rails, weighing about 75 lbs. per lineal yard, the ends being bent so as to bear against the face of the piles, which often became twisted in the course of driving through the rubble. Many

[1] Barling on Tyne North Pier Reconstruction, *Min. Proc. Inst. Mech. E.*, Newcastle Meeting, July, 1902; and *Min. Proc. Inst. C.E.*, vol. clxxx.

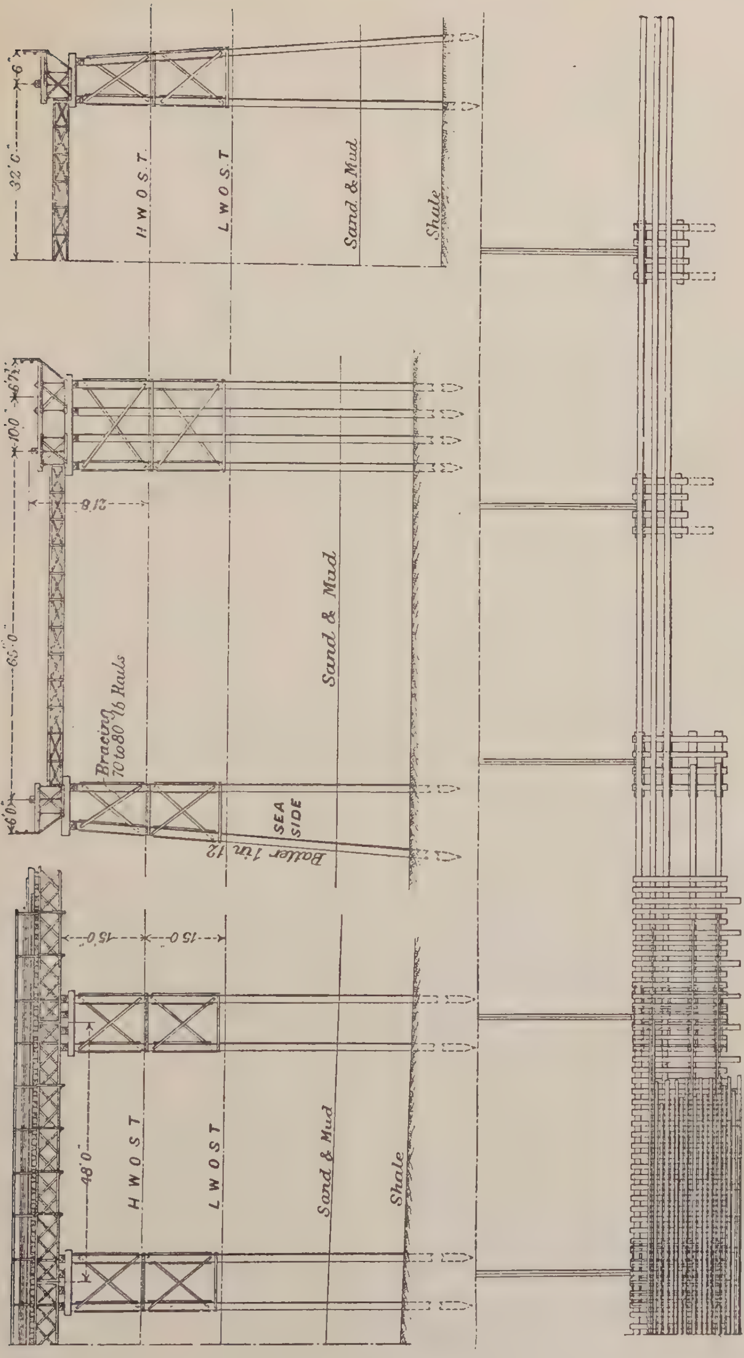


Fig. 176a.—Temporary Staging at Tyne North Pier Reconstruction.

of the rubble stones were of large size, and a shoe weighing no less than 110 lbs., with a bearing area of 56 square inches and a point of chilled cast iron, was found necessary to force a passage through. The girders between the trestles were of latticed ironwork, each span consisting of three girders on the north side and four on the south; the under sides were at a height of 15 feet above H.W.O.S.T., except in the case of the three outermost spans, where, by supporting the way-beams for the Goliath road on cross girders resting on the lower flanges of the main girders, the level was raised to nearly 19 feet above H.W.O.S.T. The object of raising the level of the girders was to keep them clear of the waves, which attained a greater height towards the outer extremity of the work. The level of the Goliath rails was in all cases 21 feet 8 inches above H.W.O.S.T. Light lattice girders spanning from side to side of the staging maintained the gauge.

The outermost Goliath operated a large Hone single-chain grab with a capacity of 115 cubic feet and a weight in air of $7\frac{1}{2}$ tons. This was employed for removing the rubble and material in advance of the work. Immediately following, a second Goliath carried a large cast-iron diving bell measuring 12 feet 10 inches by 9 feet and 6 feet high; the weight was about 22 tons in air, but only 2 tons under water, in working condition. The crew of four men were engaged in accurately levelling the bottom, which had been laid bare, in so far as the running sand would allow. This layer of sand lying above the shale gave considerable trouble, for unless the blocks could be set immediately after the bed had been prepared for them, the sand accumulated and had to be removed. Owing to this difficulty the quantity actually excavated was often ten times as much as the nominal quantity, and every cubic yard of bottom excavated occupied the bell on an average about six hours.

The blocks were set by a third Goliath. A header bond was adopted throughout, a short and a long face-block being placed in alternate tiers of the same course. The position of the foundation course was usually determined by means of 110-lb. lead plumb-bobs attached to wire lines, but owing to tidal currents these could not be depended upon for great accuracy, and at every few hundred feet piano wires, the lower ends of which were attached to the blocks already set, were plumbed by the theodolite to form a coupling between an under water survey and the standard points on the top. Owing to the water being generally thick these surveys could not be carried out very frequently. The greatest possible care was taken to ensure the tops of the foundation blocks being in one plane, and levels were taken at each corner of the block as it was set; any deviation from level was corrected by lifting the block and re-adjusting the bed. Foundation blocks of different thicknesses (3 feet, $4\frac{1}{2}$ feet, 6 feet, and 7 feet) were used to meet varying conditions.

Breakwater Construction at Alderney.—The following detailed description of the operations in connection with the construction of the wall or

superstructure of Alderney breakwater will be found extremely instructive. It is quoted from an account[1] by Mr. (now Sir) John Jackson, who acted as contractor's agent on the works for a period of nine years. The breakwater unfortunately subsequently acquired an unenviable reputation on account of the large annual expenditure incurred in its maintenance.

"In building the walls, as no machinery of any kind could remain out during the winter, the works had to be recommenced every year. The first operation was taking down a machine called the 'Samson,' invented in Alderney. This was like a railway turn-table on wheels, with balks of timber 76 feet long, placed across and over-trussed; one end, which projected past the side of the table farther than the other, was called the jib, and at the other end was the balance-weight. A double-purchase crab was fixed in the centre, which worked a chain over a travelling sheave near the end of the jib; and the whole revolved on the under frame. The gauge was 15 feet: just the space left between the travellers or gantries spanning the sea and harbour walls. This machine was capable of lifting a weight of 4 tons in the water at a distance of 30 feet from the outside edge of the turn-table. The first operation was to stretch the jib outside the end of the previous season's work, the foreman labourer standing on the outer end. This man held a copper wire attached to a large cast-iron plumb-bob, which, at slack tide, he let down to the place where the first pile or upright of the stage was to stand, and at this spot a helmeted diver excavated a hole in the bank to receive the pile, to the end of which was fastened a stone weighing 15 cwts. When the hole was excavated to the required depth, the divers retired, and the pile was lowered by the Samson into its place. Four piles were set in a row 30 feet apart, and longitudinal-trussed beams, 2 feet 4 inches by 1 foot 2 inches, were placed from pile to pile and formed a bay of staging, which consisted of 1,050 cubic feet of timber and 3 tons 8 cwts. of wrought iron in trussed rods, knees, bolts, straps, etc. The carpenters erected a bay of staging in a week. When the stage had advanced seawards three lengths or bays, six travellers or gantries, each capable of lifting 20 tons, were taken down the wall—an operation performed by the carpenters generally in one day. The gantries spanned the sea and harbour walls, and the space between was occupied by two lines of railway for conveying men and materials; and in this way the whole width of the top of the stage—viz., 70 feet—was occupied.

"About the middle of May in every year the first block was lowered by the helmet divers' gantry to its place. The helmet divers' stage was suspended by iron rods from the beams of the main stage, and hung about 10 feet below it. To this stage wrought iron ladders were attached for the convenience of divers descending to their work. Six divers were under water together—four on the seaside and two on the harbour side. They remained down four hours at a time, when there was a shift; and there were three shifts in the day. The life-line men and pumpers remained on the work all day, but the pumpers

[1] *Min. Proc. Inst. C.E.*, vol. xxxvii., pp. 87, *et seq.*

were relieved every half-hour. The divers' apparatus and the stage were removed every night, so treacherous was the sea, for even in summer it was not safe to leave anything at the level of the divers' stage; but at the height of the main stage, 10 feet higher, or 20 feet above high water, the sea seldom disturbed anything. The mode of bringing the work up was by taking advantage of the spring tides; thus it was expected of the divers that, in a fortnight, they would bring the diving work up to the level of low water, for a distance seaward of 60 feet, ready for the masons. Whatever excavation was required for the lowest course, it being a great deal more at some times than others, or however rough the sea had been, the divers never failed to prepare a length of 60 feet; but they frequently went down a second time for an extra shift to accomplish this. The average day's work of a diver in the year 1860 was $8\frac{1}{2}$ cubic yards of building; in 1861 it was 11 cubic yards; and in 1862 it was 14 cubic yards. Their work was excavating foundations, receiving the granite face-stones for the sea-wall, and setting the granite and concrete blocks of the sea and harbour walls. The stones and blocks were speedily lowered by a single chain 45 feet from the top of the stage by the gantry crabs and a rope-break. This latter was a piece of rope, with rope yarn twisted round, made fast to the frame of the crab and then fastened to the pinion shaft of the single and double gear. The chains employed were of the best charcoal iron $\frac{7}{18}$ inch thick, and they were only used for two seasons.

"The divers' work being ready for the masons, to the level of low water of spring tides, two days before full or new moon, the resident engineer gave orders to commence setting the face-stones in cement, if, in his opinion, the tide fell low enough; otherwise he could stop the works. The masonry of the breakwater on the sea face was of granite and native Alderney stone; on the harbour side it was of native stone, and the space between was filled in with backers and small rubble. The backers were as large as the machinery would lift, and were set in Medina cement and sand, in the proportion of 1 part of cement to 2 parts of sand. Sand suitable for building occurred in abundance in the island. No better cement could have been had for the purpose, for very often the masonry, ten minutes after it was built, was covered with water. No large stock of cement was laid in, as its quick setting qualities were impaired by time. The masons endeavoured to keep the work done at each tide as long as possible out of the water, and to get a course closed up in one operation. When the masonry had been brought up three courses, the tide was considered to be mastered. The helmet divers had two gantries appropriated to their work, and the masons had four. The masons came out at noon and at midnight, and it was always low water at that time; therefore, nearly half of the wall above low water was built in the night. The average day's work for each mason was $6\frac{1}{3}$ cubic yards, and no piece-work was permitted."

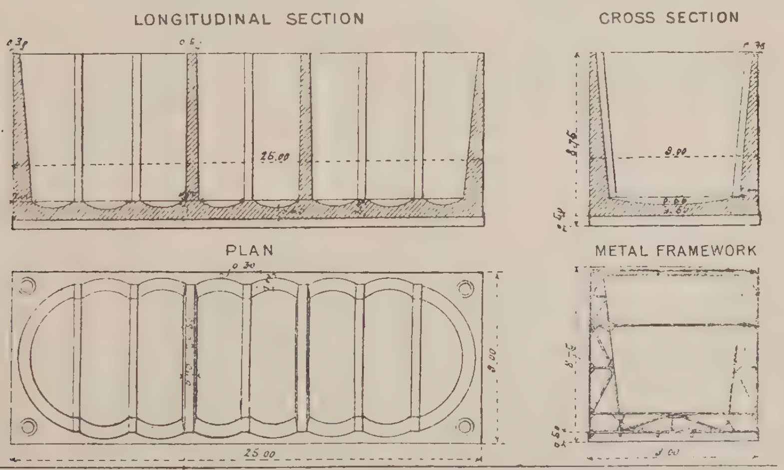
The diving work was always ended in August, and the building up to the quay level in September, while the promenade was generally finished before

Christmas. The rate of progress gradually increased from 300 linear feet to 600 linear feet of wall per annum.

Mole or Jetty at Zeebrugge.[1]—The outer portion of the jetty at Zeebrugge is constructed in solid concrete, and it has a foundation composed of monoliths, or blocks, 82 feet long by $24\frac{1}{2}$ feet wide in the portion flanking the quay, and by $29\frac{1}{2}$ feet in the portion beyond, the heights ranging from 23 feet to 30 feet, according to the level of the foundation bed.

These concrete blocks are inclosed in an iron framework caisson, which served as an outer shell. The plating in sides and floor is $\frac{1}{8}$ inch thick. Lattice beams $3\frac{1}{4}$ feet high stiffen the bottom; eighteen frames strengthen the sides; and on the underside is a rim or edge 18 inches deep.

The caissons were manufactured in the workshop adjoining the site, the



Figs. 177-180.—Caissons at Zeebrugge.

various parts being conveyed to the block-yard at the inner harbour by means of railway trucks, from which they were unloaded by an overhead land traveller or gantry.

This land traveller was a kind of movable bridge running on a double track. Its extreme span was 220 feet, the length comprised between the supports being 66 feet, and the two cantilever arms 66 feet and 88 feet respectively. It served five rows of caissons.

The caissons were put together on wood blocks or packings, but, as soon as the rivetting was finished, these were removed and the caissons lowered to the ground.

At this point, concreting work was put in hand, commencing with the floor. The concrete was composed of 3 parts of broken stone, 3 parts of sand, and $12\frac{1}{2}$ lbs. of Portland cement to the cubic foot. It was mixed by an

[1] *Vide* "Les Ports et le canal maritime de Bruges," par M. L. Coiseau. *Mémoires de la Société des Ingénieurs civils de France. Bulletin de décembre*, 1904.

electric motor mixer, served by a crane which lifted a box containing the dry ingredients and tipped them into the hopper of the mixer. Hence there was a gradual sliding progression into and through a cylinder working with a rotary movement and slightly inclined. The materials were turned over and mixed—dry throughout one-third of the length and wet throughout the remaining two-thirds, the water being administered through a central perforated tube. The output could be regulated at will.

By the time the concrete had reached the extremity of the cylinder, it was thoroughly incorporated and all the stone well bedded in mortar. It was then allowed to fall into compartments on small waggons, and conveyed along double tracks on to a traveller of the same form and range as that already described. The waggon boxes were tipped into shoots attached to the traveller, and set over the caissons.

While the concrete work of the floor of a new block was in hand, a large framework "mould-stripper," consisting of a stage resting on strong cross-beams supported by two upright frames, the same distance apart as the frames of the concrete mixer and running over the same track, removed the moulds from the blocks previously finished, with the assistance of two electrically-worked derrick cranes.

The concrete mixer next turned back and concreted the partitions, and, this being done, both it and the mould stripper were free for another block.

The blocks, as was remarked, were constructed in the inner harbour and a branch dock, which was emptied for the purpose of these extension works. As soon as the new lock and its entrance channel had been completed, water was again admitted, and the blocks, being finished, were ready to be towed to their allotted positions.

The first block was set in place on 20th May, 1900, and two others succeeded it before the end of the year. The work was then interrupted by a severe storm on 27th January, 1901, and was not resumed until 20th October following, in consequence of the damage which accrued to a framework jetty connecting the mole with the shore.

The operation of setting the blocks in position was as follows :—

In the first instance the blocks, as stored in the inner harbour, were allowed to fill with water to keep them stationary. Their sides projected from 15 to 30 inches only above the surface. When about to be moved, they were emptied by a centrifugal pump, which was inclosed in sheet iron casing to protect it from the water, for it had to be sunk below the surface to avoid the necessity of charging it. As the process of evacuation proceeded, two rows of props or stays were inserted in the caisson at right angles to the sides at each counterfort. These stays were destined to resist the external pressure of the water, the side walls of the caissons being insufficiently strong in themselves.

Next was placed on top of the caisson a stout log of timber 60 feet long,

and $15\frac{1}{2}$ inches square, and it was firmly secured to anchorages in the concrete. The object of this was to assist the alignment of the caisson.

Two longitudinal timber walings were placed along the sides of the caisson and connected by through bolts, forming transverse ties. This was necessary to strengthen the side walls in case of an excess of internal pressure, and to prevent them from yielding outwards in case of invasion by waves in a rough sea.

Finally, a huge steel cable encircled the caisson and formed a means of attachment to the tug.

The block was thus in every way equipped and ready for towage. It floated easily. The hollow spaces in its interior were adequate to counter-balance the weight of the concrete and framing, and to permit it to emerge from 2 feet to 2 feet 6 inches out of the water.

It was towed through the lock by a tug of 300 H.P. At the outer gates a second tug of the same power placed itself in tandem. In the open water a third tug joined the rear of the procession so as to prevent the block from yawing. Departure took place about the time of high water. At this time the flood current was still very strong, and the caisson presenting its broadside thereto tended to be drawn in a direction away from the jetty. About an hour and a half was necessary for each journey

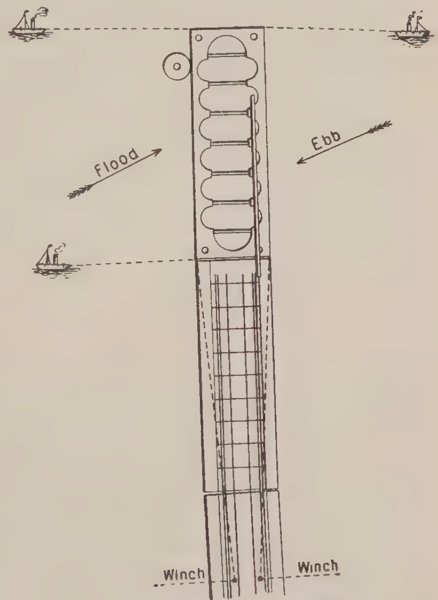


Fig. 181.—Aligning Caisson, Zeebrugge Harbour Works.

It was found that on the way the caisson often shipped quantities of water, and so wood coamings were provided to increase the freeboard and prevent the incursion of waves. This was the more necessary during that period of the work when the roadstead was very exposed. As the work progressed, more and more shelter was obtained, and the blocks were finally able to travel in perfect tranquillity.

When a block had reached the extremity of the finished portion of the jetty, one end of it was moored up thereto by cables which led back to winches on the jetty, and the flood-tide commencing to slacken, the block was gradually swung round by the tugs and brought into proper alignment. Its position was accurately determined by two guiding elements: one, on the right hand, the long beam already alluded to, and arranged so as to bear against the jetty structure; the other, on the left, at the outer extremity,

an enormous truncated cylinder of concrete 13 feet high, weighing 55 tons and resting on the sea bottom.

As soon as the slack of the tide arrived—it only lasted a little more than a quarter of an hour,—the plugs which closed three orifices in the sides of the block were removed, and water flowed into the interior of the caisson, which gradually foundered. The process of sinking was hastened by the water pouring over the side walls until the caisson finally disappeared from view amid a swirling sea. The sight is stated to have been remarkably impressive.

The caisson reappeared above the surface at low water, when 3 feet or so of it became visible. It was then freed from its temporary strutting, guide beam, and moorings.

The filling in of the compartments with concrete commenced immediately, and was continued to conclusion without cessation, except during the period of strongest tidal run towards high water.

The concrete was manufactured in a yard situated at the junction of the jetty with the shore. At this point were assembled immense banks of gravel from the Rhine, and quantities of Portland cement from Cronfestu, Niel, Tournai, and Les Laumes. The stores for the cement consisted of three galvanised iron buildings, covering a superficial area of nearly 1,100 square yards.

Four electric concrete mixers, each manufacturing 260 cubic yards of concrete per day, were located at each side of the stores; they were fed by an electric crane, and water was laid on to each from a water tower.

Suitable sidings permitted waggons carrying skips to present themselves under the shoots of the concrete mixers. The gravel and cement duly measured were deposited in layers in these skips; the crane lifted the skips, swung them, and emptied their contents into the hopper of the mixer.

The completed concrete was discharged into skips holding 12 cubic yards each. These were carried on special trucks drawn by ropes from electric capstans.

When four skips had been marshalled, a locomotive took them in charge and conveyed them to the extremity of the jetty. Here a Titan crane or a pair of sheer legs lifted them from the trucks, carried them over the caisson, and lowered them to the bottom, when they were automatically discharged without loss or disturbance. After each discharge the skip was withdrawn and replaced on the waggon.

Upon the foundation course thus prepared was raised the body of the jetty, which comprised three courses of concrete blocks, each $16\frac{1}{2}$ feet long, $8\frac{1}{4}$ feet broad, and $6\frac{1}{2}$ feet high, weighing 55 tons. The concrete of which they were composed was rather richer in cement than that previously described, in that it contained $14\frac{1}{2}$ lbs. of cement per cubic foot instead of $12\frac{1}{2}$ lbs.

[*To face p.* 244.

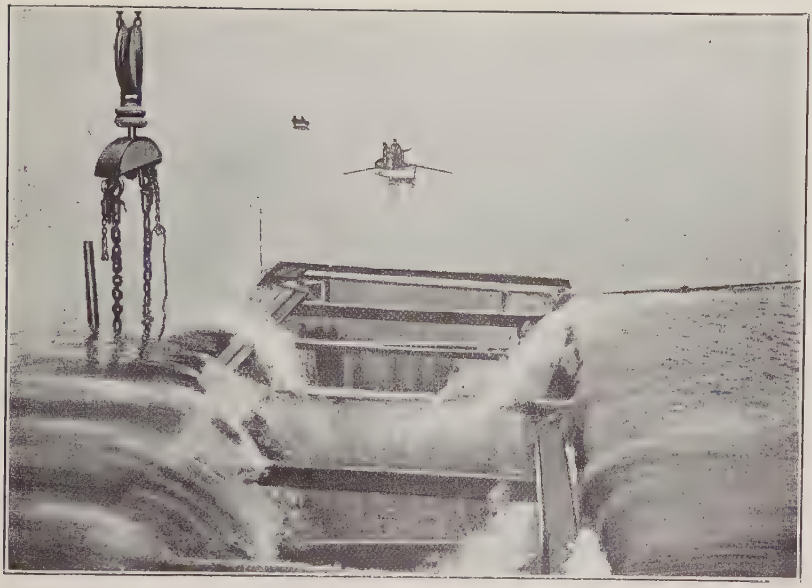


Fig. 182.—Sinking a Caisson at Zeebrugge Harbour Works.

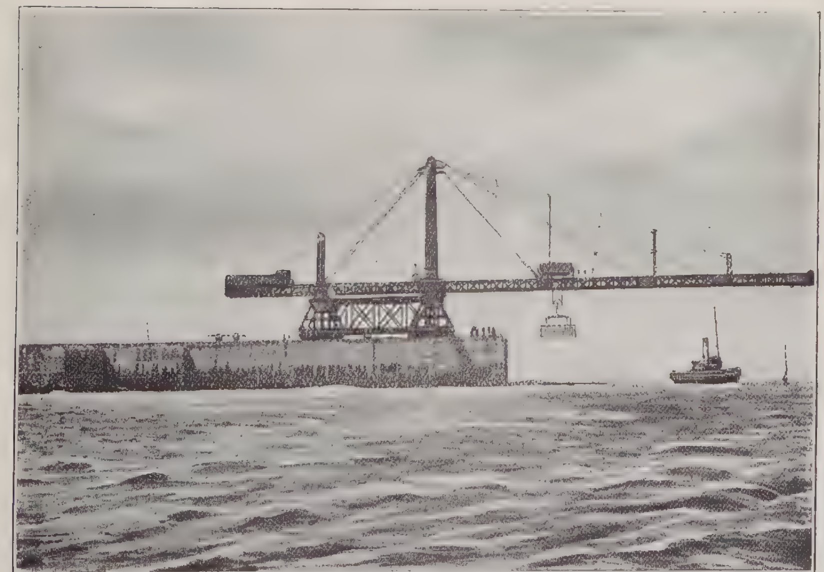


Fig. 183.—Titan at Zeebrugge Breakwater.

Breakwater at Cette.—New spurs extending the existing breakwater at this port at each extremity of its length were constructed between the years 1881 and 1895. The foundation is of a very shifty nature, being sand exposed to considerable scour. Such action inevitably tends to settlement and dislocation in any structure built upon it. Still, the problem had to be faced, and the system adopted was as follows:—Upon the treacherous base was deposited a mass of small riprap and rubble, the pieces ranging up to a weight of 440 lbs. and forming a core about 80 feet wide by 13 feet thick (fig. 184). Above this there is a layer, one-half that thickness, of larger rubble, the largest lumps of which attain a weight of 4 tons each. This layer extends seaward of the riprap core for a distance of about 60 feet, and thereon is laid, in regular horizontal courses, artificial blocks having a volume of 700 cubic feet each.

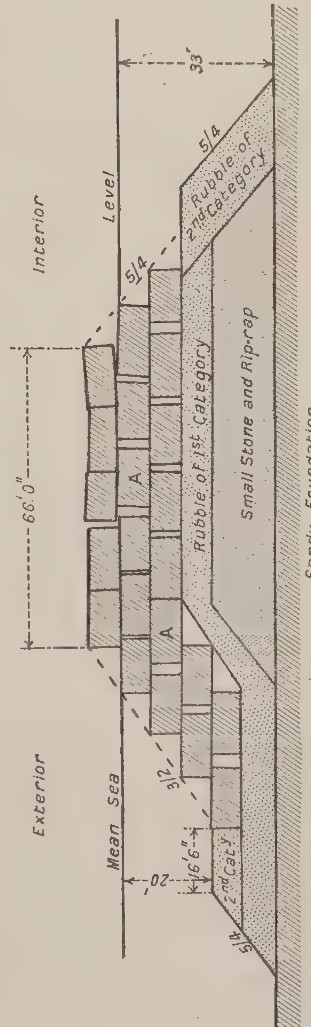


Fig. 184.—Section of Cette Breakwater.

The blocks were deposited by floating derricks, and were so placed as to have their longitudinal axes perpendicular to the line of the breakwater. They are not in actual contact with one another, but the spaces of 24 or 30 inches between them have been filled up with masonry and concrete so as to form an unbroken front.

The lowermost two courses of blocks, however, on the sea side were simply tipped into position, the upper surface being roughly levelled by means of rubble filling. Altogether, the blocks were not adjusted with the precision which is characteristic of similar breakwaters elsewhere—at Genoa, for instance.

At first it was intended to surmount the whole structure with a blockwork parapet, but this idea was abandoned, as the addition would probably have reacted detrimentally upon the stability of the outer slope by increasing the recoil of the waves. As it is, the sea overrides the breakwater in rough weather, and this, combined with an unstable foundation, produces movements in the outermost blocks. Voids are created, and these have to be filled with fresh blocks tipped as closely

as possible into position. Hence, the seaward face is losing, to a very large extent, its arrangement in regular courses, so that the desirability of creating, or of attempting to maintain, anything of the kind with detached blocks in an exposed position, is open to question.

The cost of depositing the blocks was as follows :—

Tipped overboard, 57s. per block.
Set without regular coursing, 78s. 6d. per block.
Set and coursed regularly, 92s. 4d. per block.

The cost of the breakwater complete ranged from £75 to £90 per foot run. The cost of replenishing the blocks on the outer slope forms a current charge of about eight guineas per lineal yard per annum.

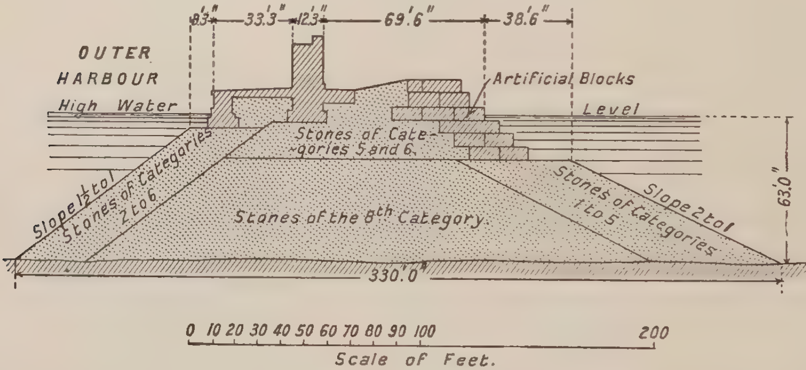


Fig. 185.—Section of West Mole at Genoa.

Breakwater at Bilbao.—The protection works in Bilbao Bay, at the mouth of the River Nervion, afford another illustration of the method of construction by caisson monoliths. The example is the more interesting in that the original design for the breakwater was very materially modified under the severe experience gained in the course of its formation.

The original design is shown in fig. 186. It comprised all the features associated with breakwaters of the mixed type—viz., an inner core of small, with an outer layer of large, rubble stone surmounted by large concrete blocks deposited at random, upon which was to be erected a superstructure of mass concrete with blockwork facings, and an upper parapet wall. The foundation consisted of mud and sand except in the parts immediately adjacent to the shore where the rock was exposed. The artificial blocks contained from 40 to 65 cubic yards each, and brought the level of the work up to low water line of equinoctial tides. Begun in 1888, the substructure was allowed to settle for a couple of years before any additional weight was imposed upon the foundation.

In 1891 the superstructure was commenced, and in 1893 indubitable evidence was given of the prejudicial, and even disastrous, influence which it

exerted upon the breakwater as a whole. The waves striking against the vertical face of the wall fell back with great force upon the top of the mound, disturbing the blocks and laying bare the rubble core, which was then easily washed away. The experience was renewed and confirmed in the following year by a storm which has already been alluded to.

Accordingly, a change of plan was decided upon. The superstructure was commenced at a level of 16½ feet below its former level. Most probably this in itself would have been insufficient to secure immunity from undermining, resulting from the collapse of waves and their back-draught, had there not been additional shelter afforded by the setting back of the line of the wall nearly a couple of hundred feet from the seaward face of the artificial blocks of the original breakwater, which latter thus constituted a sort of advance

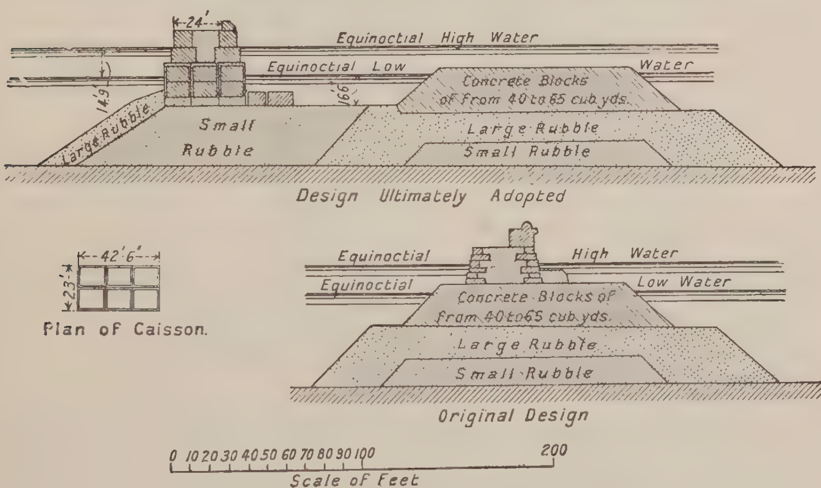


Fig. 186.—Sections, Bilbao Breakwater.

guard or outlying defence. The breakwater, in fact, was practically duplicated with a block mound in front and a wall at the rear, as shown in fig. 186. The space between the two (about 100 feet) not only reduced the force and violence of the waves, but it also afforded some constructional convenience by providing room for a tugboat to work and facilitate the building of the wall.

To render the wall as solid and homogeneous as possible, it was decided to build it with the aid of framed and plated caissons. The dimensions adopted for these were 42 feet 6 inches long by 23 feet wide by 23 feet deep. The weight of the caissons was 30 tons, and they had a light draft of 12½ inches ; but before actually towing them into position (they were constructed on the river bank), they were ballasted with a layer of Portland cement concrete 5 feet thick, which increased the draught to a little over 11 feet. When sunk in position, they afforded a margin of 6 feet 6 inches above low-tide

level. The material of the caissons was Bessemer steel in plates $\frac{1}{4}$ inch thick, strengthened internally by longitudinal and transverse bulkheads of lattice-work so as to form six equal compartments. In each of these compartments were subsequently placed two concrete blocks of about 40 cubic yards volume.

The rubble work bed on which the caissons were to rest was levelled ready to receive them through the agency of a diving-bell. The time occupied in this operation varied from one to two days. The caisson, having been ballasted, as already stated, was then towed into position during the last two hours of an ebb-tide, so as to allow as much time as possible for its adjustment and settlement. Sinking was effected by pumping water into the

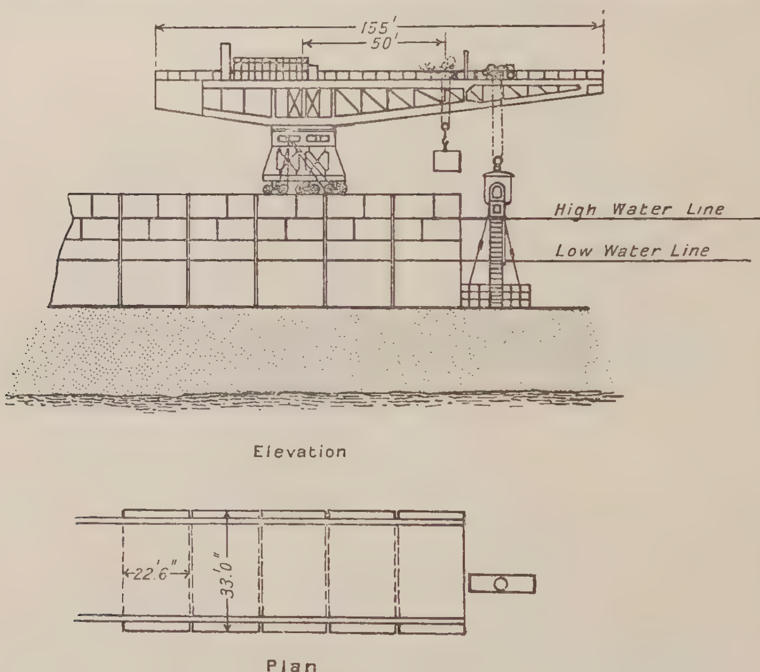


Fig. 187.—Bilbao Breakwater, showing Titan and Caisson.

interior of the caisson through a centrifugal pump suspended from a Titan crane and worked by an electric motor. If irregular settlement took place, the action was reversed and the caisson refloated until its misplacement had been rectified. Once truly placed, eight or ten 40-cubic-feet blocks were deposited in the compartments of the caisson by the same crane. This generally absorbed the whole of the time available on the tide. The following tide, after pumping out the caisson, the remaining blocks were deposited and liquid concrete was run in between them and the others with a solid layer, 20 inches thick, covering the whole.

During the third period of low water the superstructure was taken in hand. It consisted of two facings of concrete blocks, approximately 13 feet by

10 feet by 8 feet each, set in two courses as headers and stretchers alternately. The hearting was of mass concrete. Given favourable conditions and fine weather, the superstructure could be finished during the fourth tide with the exception of the parapet wall, which was not undertaken until the full measure of settlement had been obtained. This was considered to have been achieved after the lapse of a couple of winters. From the moment a caisson was first placed to the completion of the superstructure, the settlement averaged about 8 inches. Under the load and vibration of the Titan crane setting blocks further seaward, and under the influence of winter storms, a further settlement of 16 inches took place, making 24 inches in all. When this had been realised, the joints between the caissons, which were 12 inches wide, were made good with cement concrete, and the parapet wall was built.

The total amount of the contract was equivalent to £962,756, and, as the length of the breakwater is 4,757 feet, the cost works out to £203 per foot run. The quantities of material in a caisson length (42 feet 8 inches) were as follows:—

		Cub. yds.	Cub. yds.
Caisson.	Bottom ballast 42 feet 8 inches × 23 feet × 4 feet 11 inches =	178·54	
	Twelve blocks each 39·4 cubic yards,	470·88	
	Filling-in concrete,	180·20	
	Steel bulkheads and timber struts,	3·55	
			833·17
Super-structure	Eight blocks, each 39·24 cubic yards,	313·92	
	Filling-in concrete,	166·77	
			480·69
	Total,		1313·86

Say 1,314 cubic yards in all.

Breakwater at Bizerta.[1]—The type of structure primarily adopted at Bizerta for the converging jetties at the entrance to the port, built between 1889 and 1895, was the rubble mound, consisting of a core of *pierre perdue* of all sizes, surmounted and protected on the sea face by a revetment of natural blocks of large size. The site is not so exposed to violent gales as are other places on the north coast of Africa, and this system of construction was found to answer very satisfactorily. Unfortunately, the local stone (a marly limestone of poor quality, flaking rapidly in salt water) was of such a character as not to commend itself for further use, and a complete departure in design was made in dealing with the new breakwater and the extension of the north jetty, the former of which has a length of 2,000 feet, and the latter of 660 feet.

These works, as shown in fig. 188, were carried out by means of metallic caissons with movable upper works, forming ultimately huge artificial blocks faced with marble, which were laid upon a mass of miscellaneous riprap at a level of 26 feet below low water. The blocks measured 102 feet by $26\frac{1}{4}$ feet by $26\frac{1}{4}$ feet cubing at 70,000 feet, and having a weight of 5,000 tons each. They were set with great precision, and only two blocks out of twenty-three varied perceptibly from the exact line. The settlement, averaging 27 inches,

[1] De Joly on Breakwaters, *Min. Proc. Tenth Int. Nav. Cong., Milan,* 1905.

was very regular throughout. The slopes of rubble, however, intended to be 5 to 4 on both sides, had to be reduced on the sea face to 5 to 2 and a further modification of the original design was the extension of the riprap on the port side so as to afford an additional 16 feet of base. The original and modified outlines are shown in the section. The work was completed in 1903. Unfortunately, the subsequent history of the breakwater has not been free from disappointing incidents. M. de Joly reports as follows :—

" In spite of their great size the artificial blocks are not absolutely stable. A north-westerly storm which occurred in February, 1904, destroyed the regularity and symmetry of the breakwater in a few hours : not only did the blocks settle unevenly and cant slightly towards the interior, but some of them became wedged in after pivotting about their western ends. The displacement of the eastern ends amounted to as much as 6 inches, giving an angular slew of

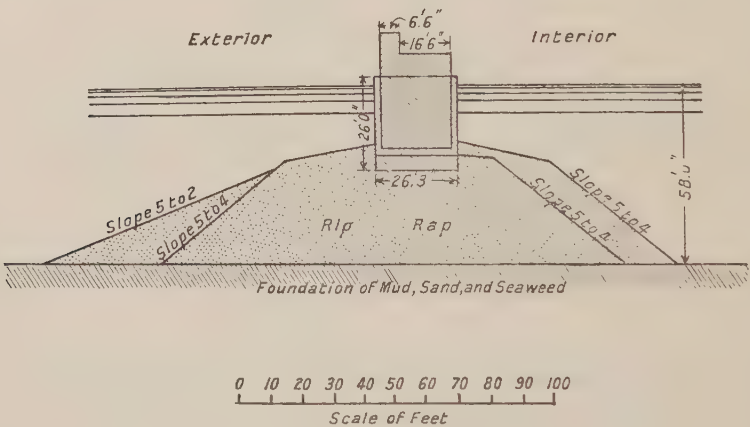


Fig. 188.—Section of Bizerta Breakwater.

" An attempt was made to avoid any return by concreting up the vertical joints, which occur every 100 feet of so ; simultaneously the interior slope was strengthened as previously described.

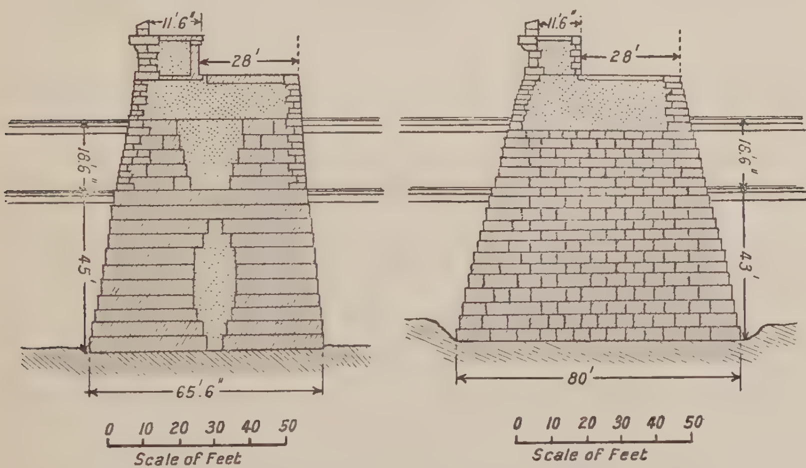
" The filling of the joints was just about finished when, at the end of November, 1904, a storm arose from the east and occasioned further important dislocations. All the joints were split and some of the blocks seriously damaged. Fears are entertained that any subsequent storm will so increase these injuries as to completely fracture the joints and split the blocks vertically.

" It seems evident from the November experience, which is different from that of the February one, that the width of 26 feet 3 inches is somewhat insufficient for a 5,000-ton block under existing circumstances, and that undoubtedly it would have been preferable to increase the width to say 33 feet, leaving the vertical joints between the blocks open so as to allow

of settlement freely without tension or shear. Also, it may be questioned whether it would not have been a better plan to make the topmost layers of riprap of harder material, which would more effectively resist crushing and grinding under the combined action of the weight of the blocks and the shock of the waves."

The cost of the earlier type of breakwater (rubble mound, with the same superstructure as that in fig. 188 above the caisson block) was £142 per linear yard, while the cost of the later caisson type was £182 per yard.

Breakwaters and Piers at Dover.[1]—The breakwaters and piers are formed of concrete blocks of from 26 to 42 tons each, according to the exigencies of the bond to which the work conforms, and the average weight throughout may be taken at $32\frac{1}{2}$ tons. The blocks were built, in the yard, of 6 to 1 concrete, and those for face-work have granite fronts. All the blocks from



Figs. 189 and 190.—Sections of Pier and Breakwaters at Dover.

foundation level upwards are bonded and dowelled with 4 to 1 concrete dowels of circular section, while above low water the courses are bedded and grouted in cement.

The blocks were set by means of Goliath overhead travellers running on temporary staging. The staging "consisted of two gantries, the distance between them being upwards of 70 feet in the clear and about 100 feet over all, in order to enable the Goliath cranes to lower into position the diving-bell and the concrete blocks forming the finished work. The timber piles, which ranged up to 100 feet in length, were of Tasmanian blue gum, the high specific gravity of which caused them to sink when they got adrift, so that there was

[1] *Vide* Matthews on Harbours of Great Britain, *Trans. Am. Soc. C.E.*, vol. liv.; The Dover Harbour Works in *Engineering*, June 28, 1907; and The Admiralty Harbour, Dover, in *Engineering*, Oct. 15, 1909.

no possibility of their damaging ships in the channel. They very satisfactorily resisted attacks by the teredo, and were used over and over again in various sections of the works. There were six piles in the dolphin or pier of each gantry, and these were at 50 feet intervals; four of the piles were driven vertically with two raking piles on the outside. Rail-struts and tie-bars were utilised for bracing the piles forming each dolphin. On the top there were girders with angle tee-pieces, and flitch timbering over them to carry the track girders, which were of lattice construction, 6 feet deep, and, of course, in 50-foot lengths. On each side there were three such continuous girders, carrying rails for the accommodation of the Goliath cranes, which had a maximum gauge of 100 feet. The dolphins were braced together by a series of lattice girders, having stiffeners near each of their ends." [1] The roads on the stagings were 27 feet 6 inches above high water of spring tides and 46 feet 3 inches above low water.

The system of operations, carried on simultaneously, was as follows:—At the outer end of the work was a stage-erecting machine, followed by a Goliath crane working a grab; then a second Goliath, from which the diving-bell was worked. Following this came a third Goliath, setting blocks under water; and behind that, a fourth, setting blocks above low-water level.

The diving-bells used on the works were 17 feet 6 inches by 10 feet, with 6 feet 6 inches headroom. They weighed about 35 tons out of water, and 5 tons when submerged. They were fitted with electric light and with telephonic communication, but it is recorded that the men preferred mechanical signals.

The foundations generally were carried from 4 feet to 6 feet into the chalk and flint bed, and were protected from the abrading action of sea currents on the outer face by an apron of concrete blocks 25 feet in width.

Excavation for foundation work was carried out by grab-dredging down to within 12 inches of the finished level, and the remaining material removed by the aid of bell-divers. Four men were engaged in the bell, excavating and finishing the foundation ready to receive the lowermost course of blocks. Each shift was of three hours' duration, and two shifts *per diem* were generally worked by the men. When the weather was favourable, work was continuous, night and day.

The greatest depth of the foundations is 53 feet below L.W.O.S.T., the average depth being 47 feet. There was thus an average working head of 66 feet at H.W.O.S.T., corresponding to a pressure of 29 lbs. This head has been found to be a maximum for working under comfortable conditions; on several occasions, when the depth was exceeded for a short period, inconvenience was experienced from the extra pressure.

Very excellent plant was provided by the contractors for the preparation of the concrete blocks. In the workyards six electric portable concrete

[1] *Engineering*, Oct. 15, 1909.

mixers were used, each capable of turning out about 100 cubic yards of concrete a day. The mixers were of the Messent type, revolved by a motor of 18 H.P., and driven from the point where the aggregate was received to the block moulds where the finished concrete was deposited, by a 25 H.P. motor. The gauge of the mixers was 11 feet 7 inches.

CHAPTER IX.

PIERHEADS, QUAYS, AND LANDING-STAGES.

Importance of Pierheads—Forms adopted—Main Features—Lighthouses and Harbour Lights—Examples of Pierheads at Madras, Toulon, Sunderland, and Pillau—Quays—Landing Slipways—Stairways and Ladders—Spending Beaches—Entrance Booms—Mooring Buoys—Landing-stages: Fixed and Floating—Pontoons—Conditions of Stability—Centre of Buoyancy and Metacentre—Case of Semi-immersed Pontoon—General Case—Case of the Ballasted Pontoon—Internal Stresses in Pontoons—Liverpool Landing-stage.

Importance of the Pierhead.—Whatever the relative exposure of other parts of a mole or breakwater, there can be little doubt that its termination—the **pierhead** as it is called—is subjected to experiences considerably more trying than any which fall to the lot of maritime structures in general. To begin with, it is unfavourably situated; and, in the second place, it is still more unfavourably adapted to its position. Exposed on three sides out of four, it is called upon to withstand and repel the most powerful subversive agencies without that uninterrupted lateral support which constitutes so important and valuable a mainstay of the breakwater proper. The absence of this support renders it a matter of the utmost moment to make a pierhead thoroughly self-sustained and independent—to treat it, in fact, as a perfectly detached and isolated structure, capable of resisting, unaided, all external influences; for in its downfall is involved the destruction of more than is contained within its own limits. The pierhead removed, the section of the breakwater immediately adjoining it has its security materially impaired, and becomes practically defenceless. It enters into that precarious condition which is inseparable from a "scar-end," and the area of damage may be almost indefinitely extended. The pierhead, therefore, should be looked upon as the keystone of a breakwater's stability.

In itself, the pierhead may possibly not exhibit to the eye any specially marked features of height or width—many minor structures do not; but whether these features be in evidence or otherwise, the necessity for greatly increased strength and powers of resistance cannot be gainsaid. As a matter of fact, most moles and breakwaters of any importance are equipped with prominent pierheads of striking outline and substantial construction. There are several reasons why this should be the case. Not only do pierheads, as a rule, project into deep water, demanding a broad and extended base to ensure corresponding stability, but they have also special functions to discharge, which require a certain individuality of treatment. They serve to mark the entrances to ports and harbours, and should, therefore, be clearly and

readily recognisable. It is to be noted, in this connection, that they also run the risk of collision and impact with passing vessels. At night-time, and in misty weather, an entrance, especially if at all narrow, should be efficiently lighted, and, to this end, pierheads are often furnished with a lantern, fixed either to a mast or mounted on a platform, or set in a lighthouse.

Pierheads, therefore, demand consideration from two different aspects: (1) as outlying works in particularly exposed situations, requiring special precautions in regard to design and construction; and (2) as a means of guidance and direction to vessels entering a port, more especially in times of stress and heavy weather.

Form of Pierheads.—With respect to the former point of view, there are, in the first place, strong reasons for conferring upon a pierhead a shape which, in plan, is symmetrical about a point or axis. Accordingly, the ends of many moles and breakwaters are expanded as already stated, and given some

Fig. 191.—Pierhead, Madras

geometrical form—circular, square, octagonal, rectangular, hammerhead, etc., as the case may be. Of these, the circular may be claimed as the most convenient shape, from the point of view of manœuvring vessels, it being easier, in case of necessity, to warp a ship round a continuous curve rather than to swivel it through an entire quadrant. The widening of the pierhead, however, somewhat interferes with the working of vessels alongside an inner quay, though, on the other hand, it serves to cut off any strong flow of current which might endanger the vessels moored there.

(1) *As regards intrinsic stability*, a pierhead should be, as far as possible homogeneous throughout, without joint or intersection. This is not always a feasible arrangement, nor in any case easy to achieve. There can be no doubt, however, that an ideal pierhead would be one of the nature of an enormous monolith. For the purpose of constructing such a monolith, a buoyant caisson chamber of the type which has already been described (p. 241) is often constructed, floated into position, sunk, and filled with concrete. Another method is to form an outer ring or boundary, of large blocks securely anchored together, and to deposit mass concrete in the interior. In this instance, it has been found essential to introduce a number

of cross ties passing right through the ring, to prevent the blocks from being disturbed under the temporary fluid pressure of the concrete.

Yet another method is the driving of an outer ring of sheeting piles, either of timber or steel, to form the necessary inclosure. But this operation is attended by some difficulties. The satisfactory alignment and driving of piles is not always accomplished with facility, even when the work is straightforward and the conditions favourable. With both these factors acting in an adverse sense, the likelihood of a successful issue is more remote. Curvilinear work in piling is a particularly awkward undertaking.

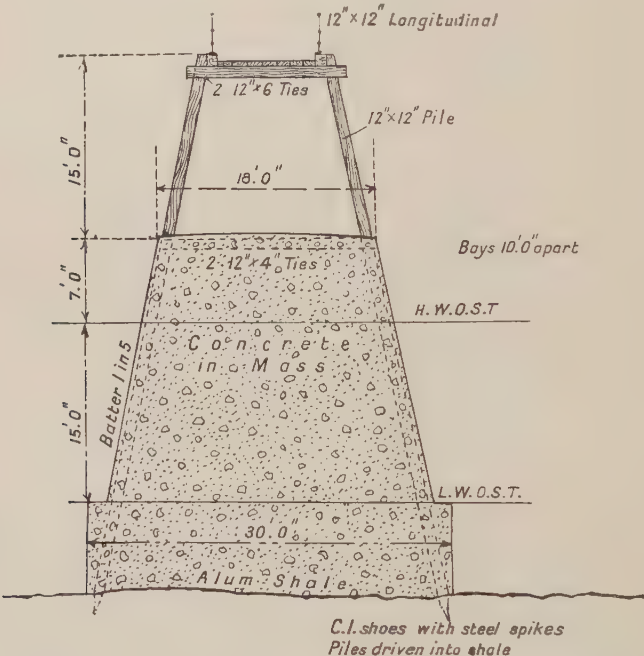


Fig. 192.—Whitby Harbour. Section of New Entrance Piers.

One word of caution is very necessary on the undesirability of forming anything of the nature of a terrace, or level platform, at the foot of the superstructure. The waves will act upon the upper surface of such a ledge with disastrous effects, both to that and to the recessed pierhead of which it is a part. The latter tends to become undermined in the manner illustrated by the pierhead at Pillau.[1] The best form of pierhead is that which presents an upright front to the sea on all sides, without benching or recesses of any description.

By making a pierhead perfectly self-contained, the problem of connecting it by any system of bond with the breakwater proper does not arise. On several grounds, and especially in reference to breakwaters liable to consider-

[1] See p. 260.

able settlement, it is desirable that pierheads should not be involved in any movement of the adjoining parts. Therefore, while the breakwater and its pierhead may be and commonly are in close contiguity, there should be a vertical joint between them, rendering them independent of each other's action.

(2) *As a means of guidance and direction* for vessels, it is desirable that a pierhead should possess some prominently outstanding features. This desideratum may be met by a lighthouse (fig. 294, p. 359), or mast (fig. 193), or, where these are not required for signalling purposes, by an elevated platform.

The **lighthouse,** if in masonry or concrete, should be stoutly and substantially constructed. Concrete work lends itself admirably to homogeneity, there being, in this case, no difficulties to encounter such as attend subaqueous work. Masonry is more costly, in that bedding and jointing demand the most careful execution; and in many cases there must be introduced elaborate bonding courses containing dovetails and keystones, all entailing considerable expense. Of the forms adopted in connection with either of the systems, the circular is, on the whole, the best, offering least obstruction to, and assisting in, the deflection of breaking seas. A flat or plane face, however, is convenient for window, door, and other openings.

The lantern being situated at the summit, the lower part of the tower should be utilised so as to afford a storeroom and also a shelter chamber for those whose duties necessitate their presence during times of storm. Stress of weather may, in fact, interrupt communication with the shore for several hours, if not whole days, at a time. A spiral staircase in the interior of the tower generally leads to the lantern.

Pierhead lighthouses may also be constructed of steel, either in the form of open framing or with plate sheeting, and also of a cluster of cast-iron columns connected by ties and bracing. Variations in design are, in fact, numerous and almost illimitable.

The **lantern** (fig. 295, p. 362) is a glazed chamber, having a pedestal of cast- or wrought-iron, and a framing of gun-metal or steel. Glazing is now generally circular rather than polygonal, as was formerly the practice. The glass is polished plate, in most cases $\frac{3}{8}$ inch thick. Spare panes should be kept handy to replace breakages, which are likely to arise from birds flying against the lantern as much as from the effects of storms.

Where the light is of minor importance and lacks the rotative mechanism which demands a protective lantern, a **mast** (fig. 193) may be used for the purpose of elevating it to the required focal plane. A steel column, tapering in form and fitted into a strong cast-iron base, will generally be used in such cases. The base must be securely bolted into the pierhead structure. At the head of the mast there will be a curved bracket, or yard arm, to support the lamp, which may be raised or lowered by a chain or wire rope, leading to a winch at the foot.

Apart from any guiding signals for the night-time, such as the above, or for the daytime, such as semaphores or other indicators, the pierhead needs to be equipped for general purposes with a capstan or two, with snatchblocks, mushrooms, or fair-leads, and also with mooring-posts at frequent intervals. Steps leading down to the water levél should also be provided in a sheltered spot. The provision of life-belts and life-lines is almost too obvious a duty to call for mention.

Pierhead at Toulon.—The pierhead of the St. Mandrier Jetty at Toulon has a diameter of 92 feet, comparable with the 42 feet width of the mole or jetty itself. Its base consists of three courses of masonry blocks, set in the form of a circle, and inclosing a space which is filled with mass concrete. The base rests upon a rubble-work deposit brought up from the sea bottom to within 14 feet of the surface of the water, and levelled with great care by the aid of divers. The superstructure is of masonry, and consists of a lighthouse with a cellar compart-

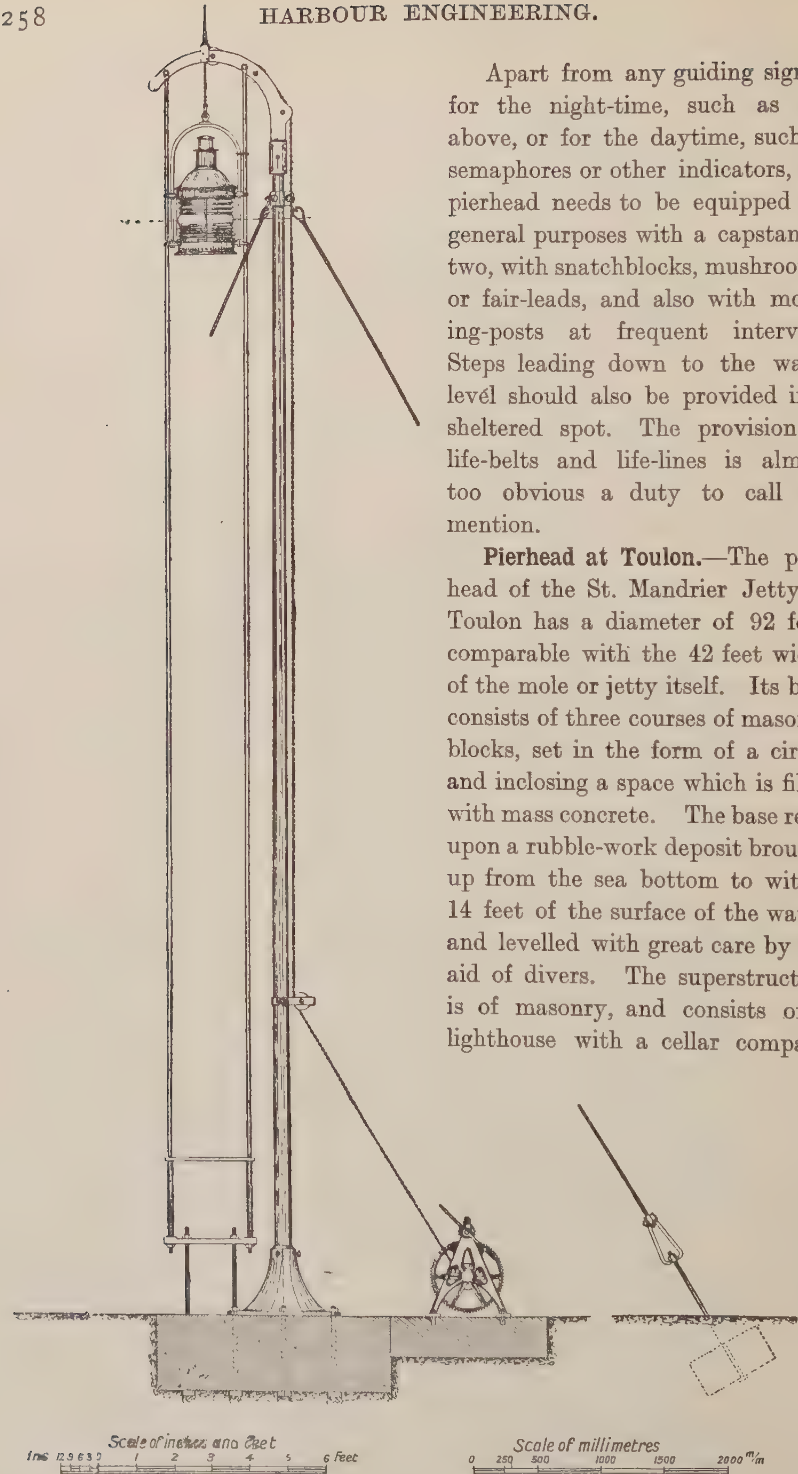


Fig. 193.—Port Light and Mast.

ment used as a cistern, absolutely water-tight, so carefully was the concrete work carried out.

The pierhead does not actually join up to the jetty—that is to say, there is no interlocking bond. A narrow vertical joint separates the two structures so as to remove all possibilities of fracture arising from unequal settlement.

Pierhead at Sunderland.—The pierhead of the Roker pier was formed by means of an iron-plated caisson $100\frac{1}{2}$ feet long, 69 feet wide, and $26\frac{1}{2}$ feet deep. The caisson, containing 3,500 tons of concrete, was floated out into position with a draught of 22 feet on to a carefully levelled foundation bed of concrete bags finished at 23 feet below low water. After being sunk, the caisson was built up with 15-ton and 25-ton blocks, mass concrete, and cement-grouted rubble, until, when completed, its weight amounted to 10,000 tons. On top of this, the pierhead superstructure was constructed in blockwork and surmounted by a lighthouse.

Pierhead at Pillau.[1]—The new moles at Pillau were built, during the closing quarter of the nineteenth century, to a type which is common on the Baltic seaboard—viz., that of a rubble mound confined between two lines of sheet piling connected by iron ties and having a brickwork superstructure. The width at mean water level between the sheet piling is 31 feet, the summit width 26 feet, and the height above mean water 10 feet.

Both mole ends have occasionally to withstand very violent attacks by the sea. The pierhead structures, therefore, were given enlarged dimensions, the width being increased to 46 feet. In plan the termination of the north mole exhibits a return in a straight line ; at the southern mole, the pierhead front forms three sides of a regular hexagon. The superstructures are set back from the ends of piled work by 18 feet in the case of the north mole, and 30 feet in the case of the south mole. The area of the recessed portion in each case was paved over with brick to a thickness of 3 feet, forming a sort of terrace or platform. The depth of water at the pierheads was 30 feet when building operations were commenced. As the piles were driven down to a depth of 50 feet below water level, no special rubble apron was deemed necessary.

The south molehead was only just completed in the year 1885, when, in the month of September, it had to withstand the onslaught of a severe gale, which blew for two days from the south-west and north-west, gradually increasing in intensity. The whole front portion of the head, the terrace work, and a large portion of the sheet piling, were destroyed. Moreover, the rubble filling inclosed by the lastnamed, in default of restraint, fell away, depriving the superstructure of its support to such an extent that it leaned forward 4 inches out of the vertical, and a deep fissure was formed at its junction with the mole proper.

The cause of this collapse, which has proved most disastrous in its consequences, is to be found in the great distance separating the superstructural

[1] Anderson on Prussian Breakwaters, *Proc. Int. Nav. Cong., Milan,* 1905.

from the substructural ends; in other words, it was due to the terrace. This was alternately attacked from below by the pressure of the water penetrating

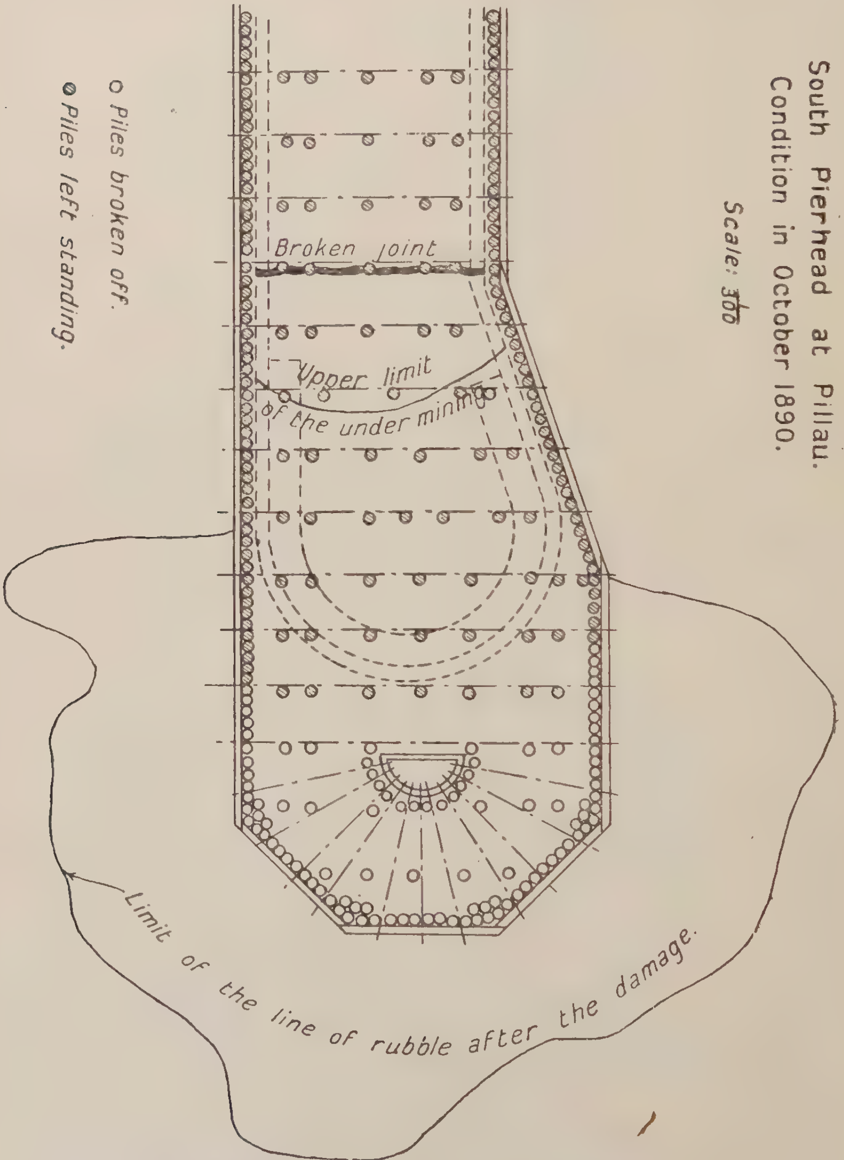


Fig. 194.—Plan of South Pierhead at Pillau.

through the openings between the piles, and through the interstices in the rubble work, and from above by the direct downward stroke of the waves. The floor was not in itself strong, and its great extent did not add to its

stability. There must also be taken into consideration the fact that the continuity of the pavement had been broken by trenches cut to receive

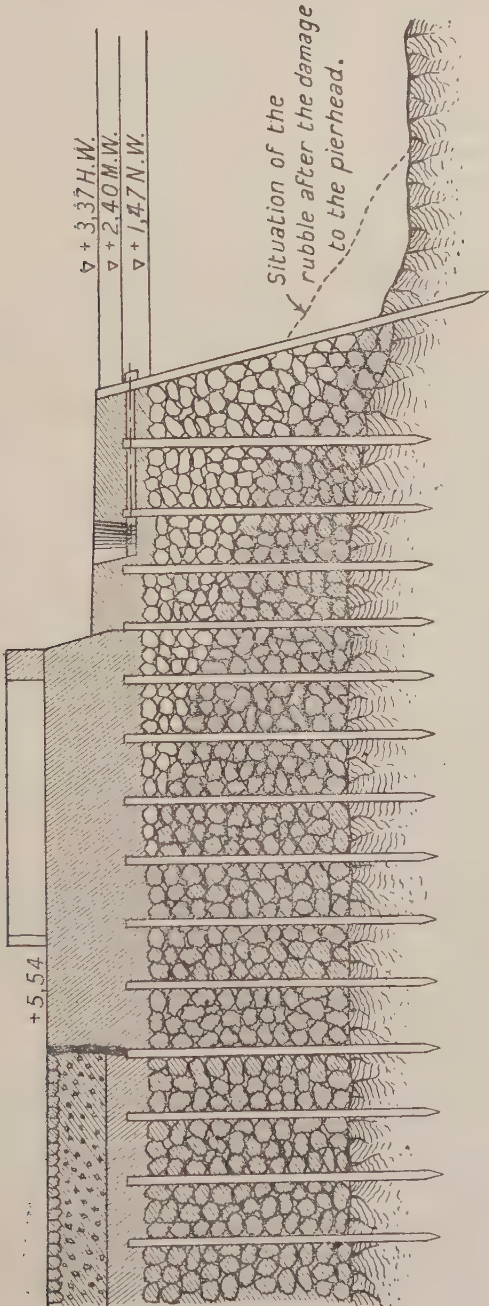


Fig 195.—Longitudinal Section of South Pierhead at Pillau.

the anchorage bars of the head. Moreover, there had probably been some settlement in the rubble work which it covered. All these causes combined to render it an easy prey to the storm, which swept away the floor and battered down the piling, with the aid of the loose masses of stone comprising the hearting.

With the front portion of the pierhead reduced to a mere mass of débris sloping downwards to the sea bottom, it became necessary to devise measures

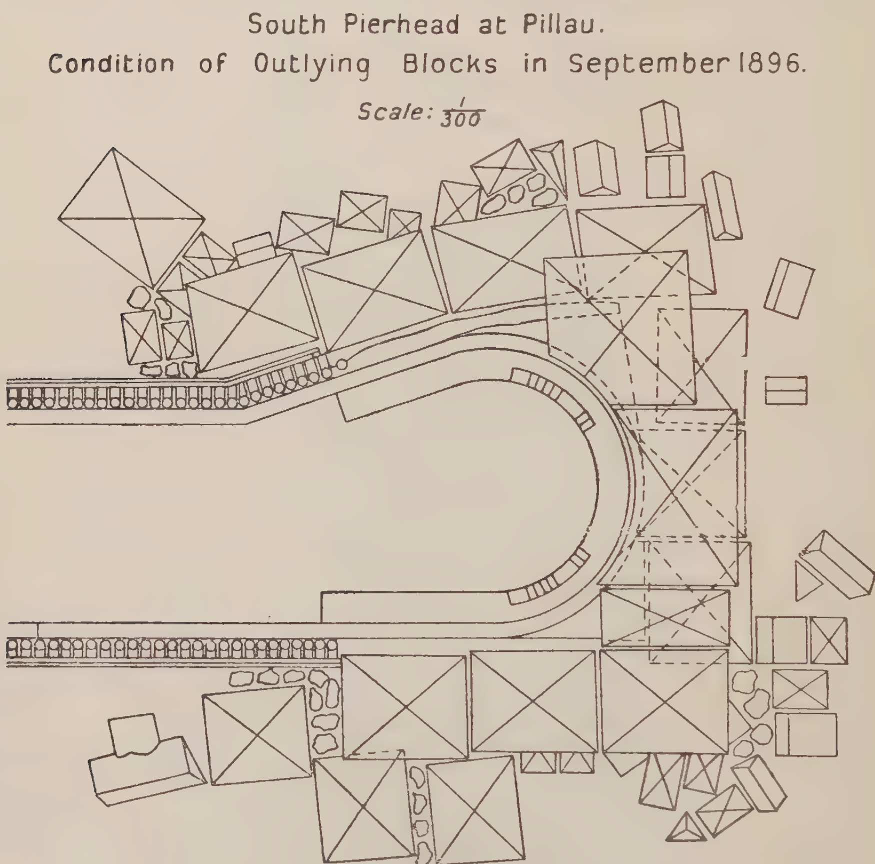
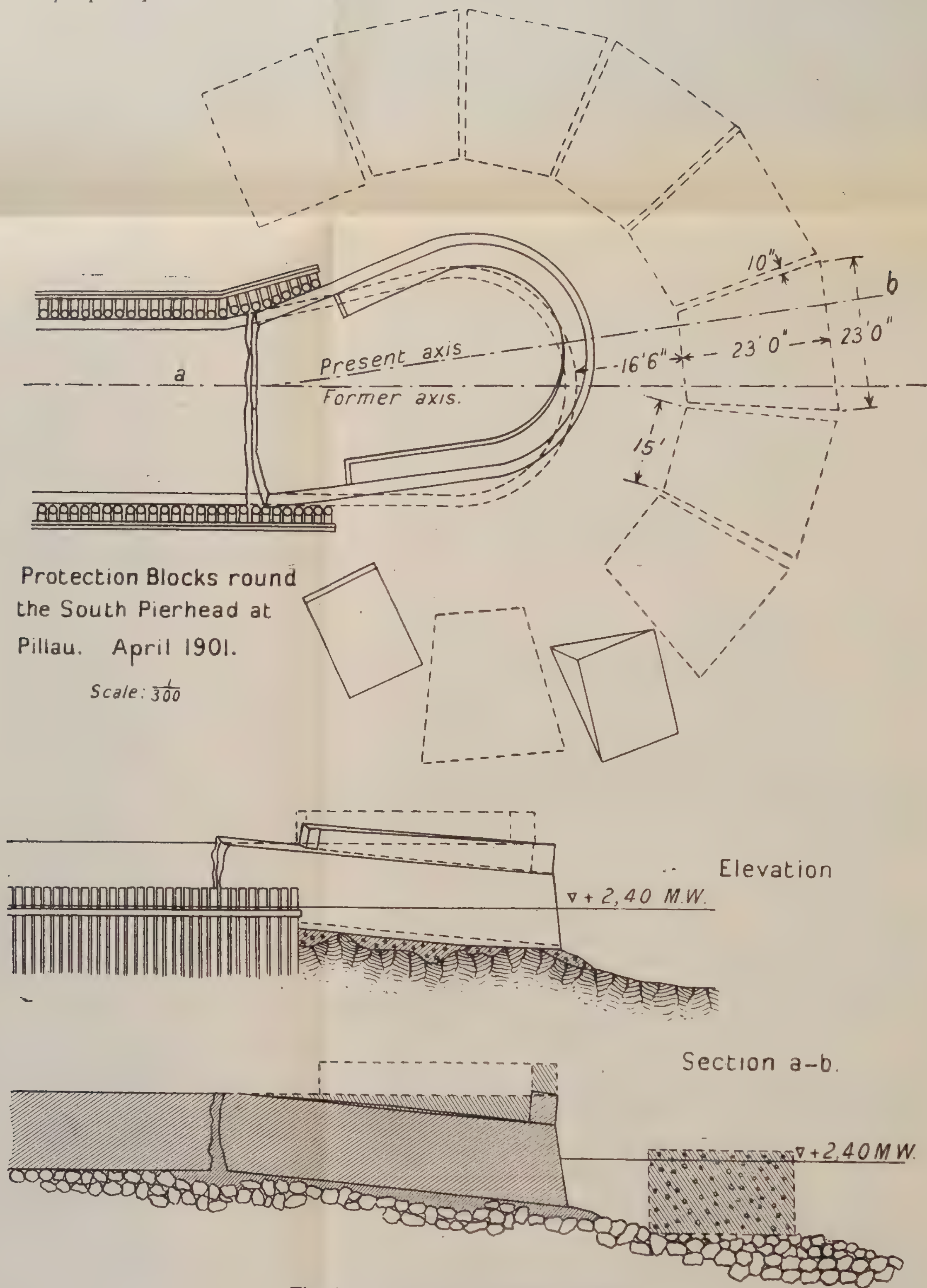


Fig. 196.—Pillau Pierhead—Protection Blocks.

for the protection of the superstructure. Any renewal of the destroyed portion was out of the question, owing to the impossibility of driving piles through the rubble mound. The plan decided upon was to mould and consolidate the mounds into a protective apron.

The first step taken was to deposit a quantity of massive granite blocks on the site of the terrace. The level at this point was 3 feet above mean low water, and it was annually increased. Up to the year 1891, over 140,000 cubic feet of stone, bulking from 14 to 35 cubic feet a-piece, were

To face p. 263.]



Figs. 197 to 199.—South Pierhead at Pillau.

deposited and formed into a berme 10 feet in width, with a slope of 2 to 1. In order to secure this stone-work in position, large blocks, containing 175 cubic feet each, were placed on top. The blocks were made of granite fragments and cement mortar in the proportion of 1 to 3 ; they had a length of 6 feet 6 inches, a width of 5 feet, and a height of 5 feet 6 inches, and they weighed 12½ tons.

Work proceeded on these lines from 1892 to 1894. A storm in November, 1894, however, once more wrecked the pierhead, destroying the uncompleted berme and its covering. All burrs and blocks lying in less than 7 feet 6 inches of water were dislodged, and some of them swept up the slope.

Subsequent measures for the protection of the pierhead have been directed to the consolidation and maintenance of the mound, but on somewhat different lines, experience having indicated that the largest blocks formed the best covering material. Thus, in addition to a large number of the smaller blocks which served to fill up gaps and to provide a flat surface, in 1895 eleven blocks of 635 cubic feet, and in 1896 ten blocks of from 530 to 1,765 cubic feet, were bedded on the rubble work in concrete composed of granite and cement mortar.

Yet these measures were attended by no better success than their predecessors. The very largest blocks, weighing 120 tons, were sooner or later dislodged by storms and driven down the slope. They were continually restored, until finally, in November 1899, a gale wrought such serious havoc that even the lower portion of the pierhead foundation was exposed and partially withdrawn. As a consequence, the whole weight of the superstructure came to rest upon the piling and staging which had been erected for pile-driving machines and rail-tracks during the construction of the mole, and which had been left buried in the mound.

The only plan now was to fill up the cavities, and this was done by means of stones and sacks of concrete packed within a circumscribing ring of blocks of 350 cubic feet, each set in a double row. But, before this work could be completed, fresh disasters occurred. The blocks were disturbed, and some of the rubble carried away, in November 1900. No satisfactory repairs could be effected during the winter season, and in the following April a violent westerly gale once more devastated the whole pierhead. The staging-piles broke ; the crack which had formed at the junction of the mole and pierhead at the time of the initial destruction of the terrace, widened out on the harbour side to a width of 4 feet 3 inches, and the pierhead, a huge mass of 23,660 cubic feet, and having a weight of 1,680 tons, was slewed on its axis through 12 feet towards the sea, and left with its outer edge depressed to the extent of 4 feet 4 inches. In this condition it rested upon a few projecting peaks of the mound.

During these experiences, it was observed that a concrete block of 1,553 cubic feet, which had been set on the deep-water side of the head, had not

suffered more than a very slight displacement. It was, therefore, decided to surround the pierhead with a ring of nine concrete blocks of extremely large size. These were formed on the buoyant caisson principle. Wooden boxes of grooved and caulked planking, 7 inches thick, were formed, at a timber-yard $1\frac{1}{4}$ miles distant, to a trapezoidal shape with widths of 23 feet and 15 feet, a length of 23 feet, and a height of 13 feet, so as to inclose a volume of 5,650 cubic feet. Stiffened by transverse and longitudinal ties, and filled to a depth of 3 feet with granite rubble concrete, they were floated to the site of the head and there sunk close alongside one another, in about 11 feet of water, a level base having been prepared for them by blasting away the protuberances of the mound, and filling the hollows with stones, débris, and sacks of concrete. The boxes were then filled up with concrete in a couple of days, in this way attaining a weight of 400 tons. Of the projected nine boxes, six were delivered in the year 1901, and, in addition, four blocks of 370 cubic feet were set. The filling of the inclosed space was then put in hand.

But in September of the same year a storm produced a fresh cross fracture in the pierhead, and a further sinking of its front face by 8 inches. The concrete boxes were slewed about 18 inches, but without damage. In the succeeding December a further and more pronounced depression of the head took place, the settlement amounting in all to 6 feet 6 inches.

By this time, however, the superstructure had found a firmer bearing on the foundation stonework. Such small cavities as remained were closed with concrete. During the following year three more concrete boxes were deposited, and the space between the boxes and the head densely packed with large stones. Owing to these improvements, the pierhead suffered but little during the gales of December 1902 and February 1903, which almost destroyed the north breakwater. The boxes, indeed, were considerably shifted, but they did not suffer much damage. The pierhead was again built up to the height of the original summit, and the breaches which had been made between the boxes were closed in 1903 by four additional boxes, and the remaining gaps were made good in 1904.

At this point the interesting but calamitous narrative ceases. It would be too sanguine a view to conclude that perfect stability had at last been attained. But, as a record of disasters and of remedial expedients, it is most instructive, and the lesson it enforces is complete.

Quays.

The inner sides of breakwaters and the shore frontages of harbours are commonly bordered with quays for the reception of merchandise and passengers.

A **quay** is, properly speaking, a paved space or area devoted to the purposes of loading and unloading craft, and it is usually bounded at the

water's edge by a wall or wharf founded at a sufficient depth to permit of vessels lying alongside.[1] A quay and a quay wall are commonly treated as synonymous terms, yet this is not strictly the case.

Quay walls are called upon to discharge very important duties. They act as retaining walls to uphold the solid material which forms the foundation upon which the quay surface is prepared and laid. They also serve to present a uniform frontage against which vessels may be moored and warped without damage. As retaining walls, they possess characteristic features and discharge special functions which call for very detailed treatment; but as the author has subjected them to a careful investigation elsewhere,[2] it is superfluous to attempt any further, and necessarily restricted, allusion to them here. Fig. 200 is a typical section through a river quay wall.

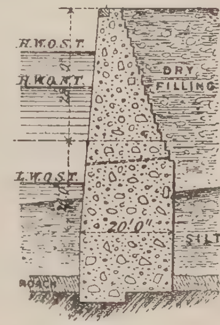


Fig. 200.—River Quay Wall at Tranmere.

The term "wharf," also used as a variant for quay, is now more commonly employed to indicate an openwork structure of piling and framing, forming the frontage of a water side embankment (fig. 201). It may, however, be detached from

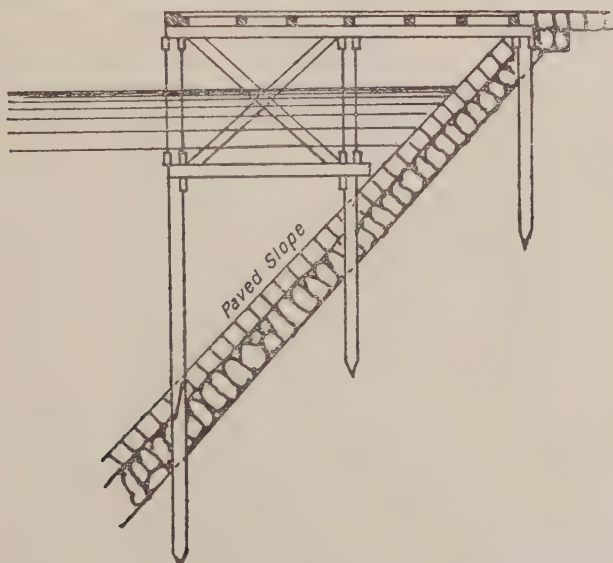


Fig. 201.—Quay and Wharf.

the shore line and approached by a gangway or arm, in which case it partakes more particularly of the nature of a jetty. If the quay is only

[1] Vessels do not always remain afloat; in many cases, with the recession of the tide, they take the ground.

[2] *Dock Engineering*, Chapter V.

required in connection with small craft, it may be faced with a sloping revetment as in fig. 201*a*.

Landing Slipways.—In situations where the water does not maintain a fairly uniform level, it becomes necessary to provide sloping ways, or slipways, leading from the surface of the quay down to the lowest water level. These slipways are even desirable under any circumstances, and especially for the purpose of affording access to small craft of shallow draught and rowing-boats, and to facilitate the withdrawal of the latter from the water. Slipways range from about 5 to 15 or 20 feet in width, with an inclination not greater than 1 in 5. They should have a covering of concrete or be paved

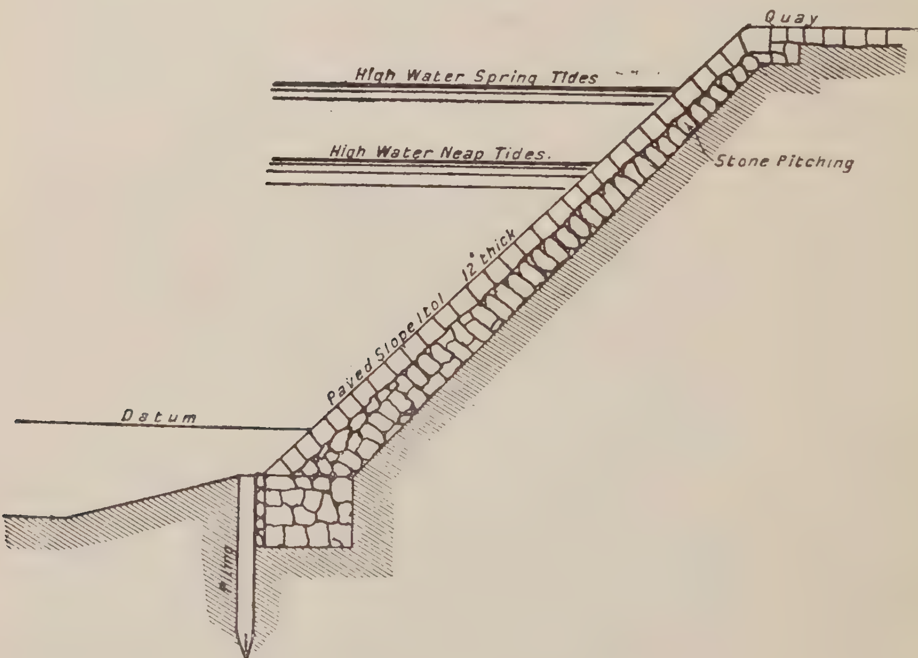
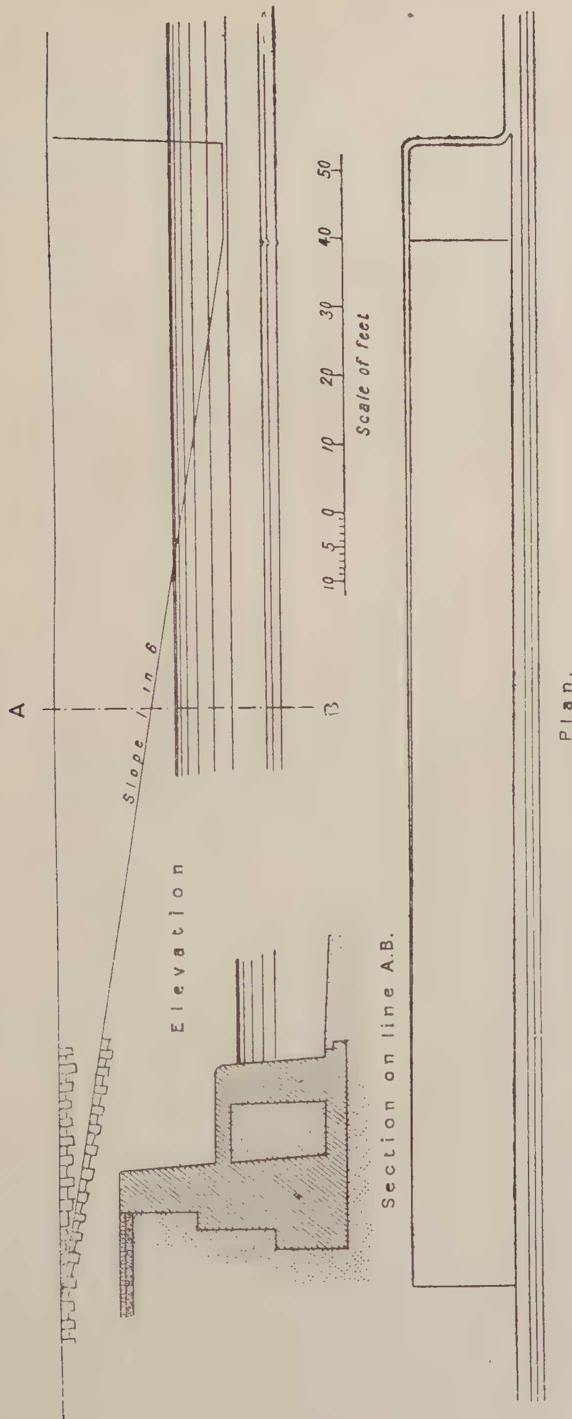


Fig. 201*a*.—Quay with Sloping Revetment.

with large heavy stones, having a flat upper surface and presenting as few joints as possible to the pick-like action of the waves. Such joints as there are should be well flushed and pointed with cement, for it can be readily understood that slipways are subjected in a peculiar manner to the most destructive action of breaking waves, which renders it imperative to present thereto as hard, smooth, and unbroken a surface as possible. Where slipways are paved with cubes, or setts, these latter must be thoroughly grouted and bedded in a perfect manner upon a substantial concrete foundation.

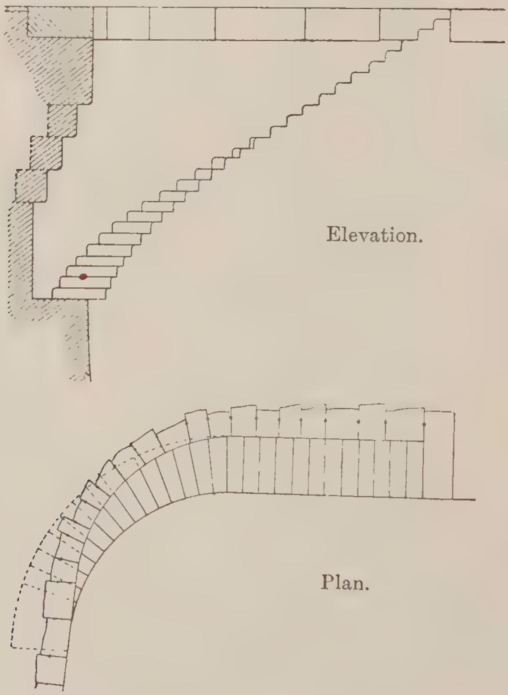
Slipways are provided with a bottom landing, and sometimes with one or two intermediate landings—all level platforms.



Figs. 202 to 204.—Slipway

Stairways and Ladders.—Access to the water-line may also be obtained by stairways and ladders. The former are simply steps set in the wall in the ordinary manner, and not uncommonly at corners, where they are least likely to interfere with the use of the quay by shipping. The latter generally consist of galvanised wrought iron vertical sides and circular rungs, recessed within the face line of the quay so as to avoid forming anything of the nature of a projection likely to produce damage.

Other adjuncts of a quay are **life-chains, mooring-rings,** and **mooring-posts.** The first-named are suspended at intervals and festooned, so as to enable persons accidentally immersed to support themselves until succoured.



Figs. 205 and 206.—Steps in Quay Wall.

The last two are for the purpose of holding vessels close against a quay wall. Mooring-rings, useful chiefly for very small craft, are now almost obsolete. as they are awkward of access and difficult to maintain in order. The most conveniently arranged of them are recessed within the quay face, so as to acquire a certain amount of cover. The best type of mooring-post has a lip arranged on the side furthest removed from the quay edge, so as to hold the rope well and keep it from slipping upwards.

A good sloping **beach** is very desirable in the immediate proximity of a harbour. If situated at the entrance, it forms a very useful spending-ground whereon waves may dissipate a very large proportion of their activity. A

beach is also desirable in that small craft may ground thereon for repairs. When formed artificially, as is sometimes necessary in rocky localities where the shore descends abruptly, quarry refuse and débris may be used for the purpose. A slope of 1 in 10 or 12 will be found most serviceable.

Booms.—Inner basins, or harbours of the smaller class, may be still further protected from the effects of sea swell by means of a temporary closure or boom across the entrance, which would naturally, in such a case, be narrow. Booms take the form of a log partition, set in horizontal layers one above another, with their ends engaging in grooves specially constructed at each side of the entrance. It is necessary to observe that unless the logs extend to the very base or bottom of the passage-way, wave motion will be transmitted beneath them, and they will prove ineffective for the purpose in view.

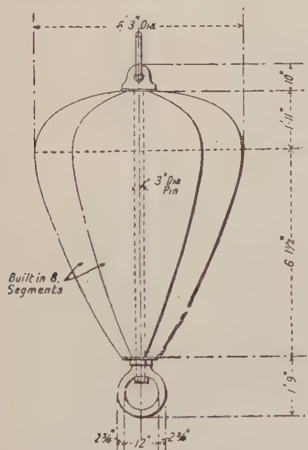


Fig. 207.
Pear-shaped Mooring Buoy.

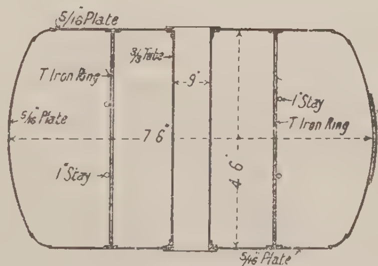


Fig. 208.
Barrel or Cylindrical Mooring Buoy.

Mooring Buoys.

In addition to the facilities afforded by posts, rings, and bollards on the quays for securing vessels, in large basins and in rivers floating and fixed moorings are also provided. The former consist of buoys of various shapes anchored to the bottom of the basin or the bed of the river, and the latter of piled stages suitably braced. These last are sometimes known on the Continent as "Ducs d'Albe."

Mooring buoys exhibit some considerable diversity in design. The four most important types are the pear-shape or peg-top; the barrel or cylindrical; the drum or cheese; and the spherical. Their outlines are shown in figs. 207, 208, 209, and 210. The first type is perhaps the most generally serviceable, combining great carrying capacity with marked stability, while its form admits of a considerable degree of rigidity, so rendering it less liable to damage. Cylindrical buoys, on account of their readiness to heel over, constitute

convenient and easy attachments for river craft. Drum buoys present a smaller surface to wave action than cylindrical buoys, and are consequently suitable for less sheltered positions. The flat top is also very convenient, especially in the case of heavy moorings, as it affords standing room for several men who may be engaged in making fast. The spherical buoy has

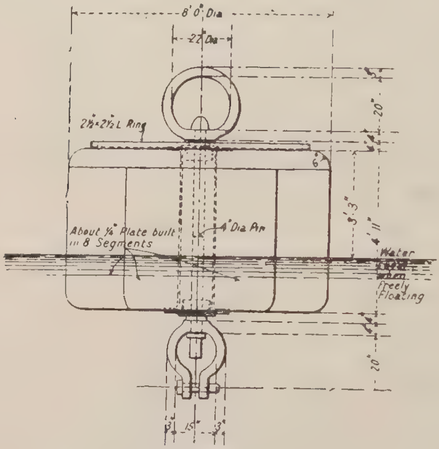


Fig. 209.
Drum or Cheese Mooring Buoy.

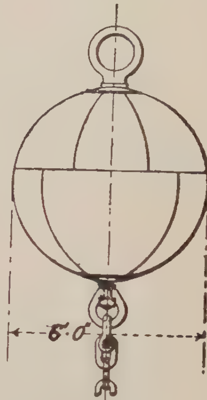


Fig. 210.
Spherical Mooring Buoy.

considerable carrying capacity, but its shape renders it more expensive to construct.

It should be borne in mind that buoys have to be designed to support, not only their own weight, but that of the cables by which they are moored.

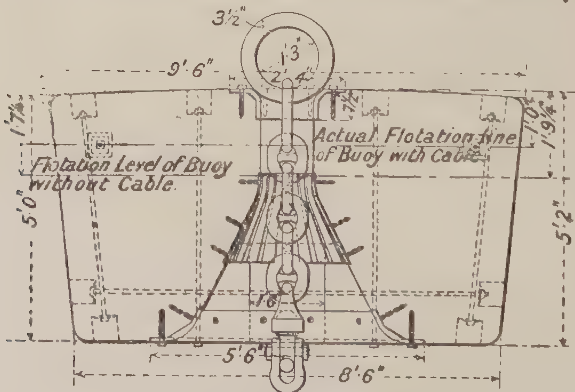


Fig. 211.—Wooden Mooring Buoy.

The amount of pendant cable will often vary considerably in tidal waters. The buoy structure should be fitted with stout wooden fendering at the water line for protection in case of impact; bulkheads are often provided as a further precaution against collision. The mooring chain is either carried through a central tube (fig. 211) or else attachment is made to a mooring

[To face p. 270.

Fig. 212.—Mooring Buoys.

bar (fig. 209). The ground attachment frequently consists, where practicable, of a screw or screws (fig. 213), but a heavy mass of concrete, stone, or iron, or other means of anchorage will serve. Screws should be buried to depths which will remove any risk of withdrawal under strain — about 10 to 15 feet will generally be found necessary. Where weight is relied upon for anchorage, 1 cwt. should be provided for every fathom of ground chain.

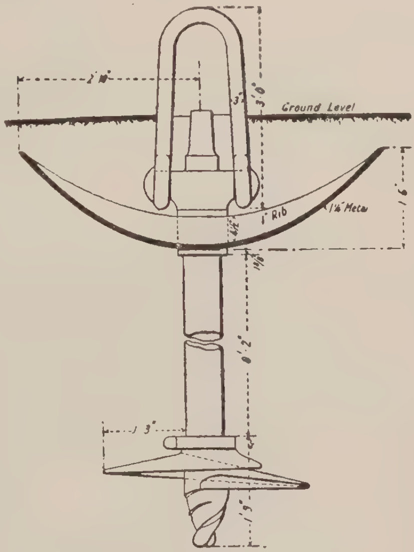


Fig. 213.—Screw Mooring.

The size of a mooring buoy and the dimensions of its fittings depend naturally on the duty to be performed, and will vary with circumstances. As some general indication, the following data will serve :—

Tonnage of Vessel.	Diameter of Mooring Chain.	Length of Ground Chain.	No. of Screws.
	Inches.	Fathoms.	
500	$1\frac{3}{4}$	35	1
2,000	$2\frac{3}{8}$	70	2
4,000	$2\frac{3}{4}$	90	3
7,000	3	115	4

Fig. 214 gives an illustration of a mooring arrangement adopted at Madras Harbour.

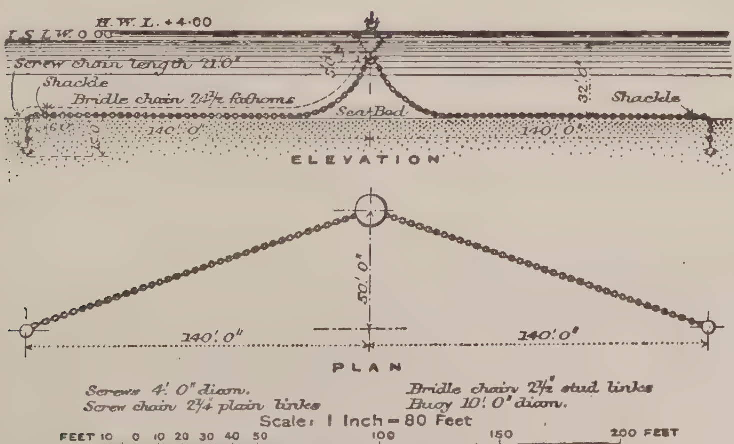


Fig. 214.—Mooring in Madras Harbour.

Landing-stages.

In all his operations the maritime engineer is more or less in touch with the requirements of the naval architect, and the boundary line between the two professions is by no means easy to define; indeed there is oftentimes a zone within which both practitioners find a common field of action, and where

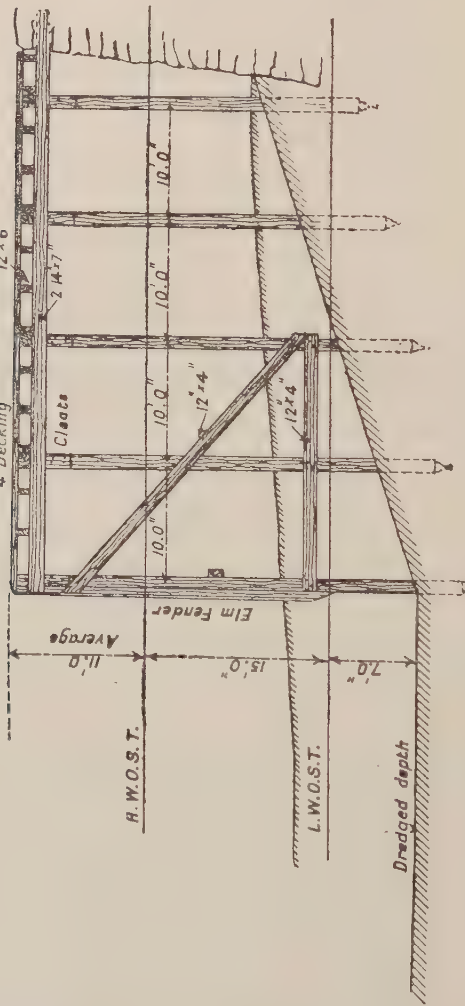


Fig. 215.—Wharf or Landing-stage at Whitby Harbour.

it would be difficult, if not absolutely impossible, to lay down any limitations for one or the other. Thus, in the case of entrance caissons, floating docks, and buoyant structures generally, there are presented to the engineer all the processes and features characteristic of ship design and calculation, and so, too, in connection with the subject of the present chapter, the laws governing

the behaviour of floating bodies have to be carefully studied and clearly understood.

Landing-stages, whether adapted for the use of passengers, vehicles, goods, or cattle, fall naturally into two main types—fixed and floating.

Fixed Landing-stages.—Besides serving no apparent purpose, it would be difficult to make an exact differentiation between the functions of a fixed landing-stage and those of a wharf, pier, or quay. Generally speaking, the term stage is limited to framed structures of timber or iron, and fixed landing-stages are most commonly platforms of woodwork on a piled foundation. But even with this limitation, they possess no features which are not common to jetties and wharfs of similar construction (*vide* fig. 215).

Floating Landing-stages are decks or rafts of timber or iron work, either self-supporting or carried upon hollow pontoons which afford the necessary bearing power by reason of the buoyant properties of water. The pontoon is the more usual arrangement, and it is certainly the only system applicable to stages of size and importance.

Pontoons.

The stability of any statical structure depends, in the first instance, upon the equilibrium of the external forces which act upon it. In the case of a floating body, these external forces are essentially and practically two, and two only—viz., (1) gravity, due to the intrinsic weight of the body, acting vertically downwards, and (2) buoyancy, or the vertical component of hydrostatic pressure, acting vertically upwards. It is true that, in addition, there are the horizontal components of hydrostatic pressure, but, as in still water, whatever the shape and size of the body, these components must always and exactly neutralise one another, there is no necessity to bring them into consideration.

We have, therefore, only to take into account two resultant forces, opposed to one another. In ordinary statics, equilibrium would be sufficiently assured by the fulfilment of the conditions, that (1) the lines of action must coincide, and (2) the forces must be equal in magnitude and opposite in direction. As regards floating bodies, these requirements are satisfied when the vertical line through the centre of gravity passes through the centre of buoyancy (or centre of gravity of the displaced fluid), and when the weight of the body is just equal to that of the fluid displaced.

But the extreme mobility of water and the absence in a floating body of any perceptible inherent resistance to disturbance, involve another phase of equilibrium in its practical and working sense. It is obvious that there must be not only perfect balance at any instant, but also a disposition on the part of the body to right itself, or to recover its initial position, in the event of a slight or moderate displacement. Both these points have to be taken into consideration in the design of floating vessels.

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The calculations necessary for the purpose are much simpler in regard to pontoons than they are in regard to ships and other navigable craft, since the former are generally constructed to some regular geometrical figure which permits of the easy determination of its centre of gravity and also of the centre of buoyancy. The calculation of the weight of an ordinary ship and the point at which it may be assumed to be concentrated, as also of the displacement and its geometrical centre, are matters of great complexity and difficulty, calling for the exercise of no little patience, ingenuity, and skill.

Pontoons, on the other hand, are generally, if not universally, either rectangular, cylindrical, or spherical in form, with centres of gravity and displacement readily determinable by simple geometrical construction.

Thus, in fig. 216 a rectangular pontoon is shown partly immersed in water. The disposition of the principal resultant forces is that shown by the arrows, and the primary condition of equilibrium is manifestly fulfilled.

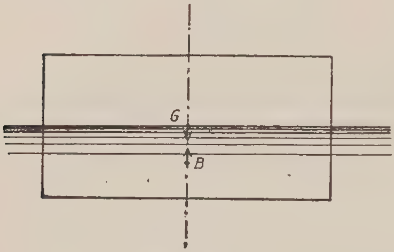


Fig. 216.

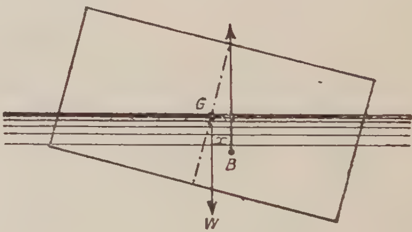


Fig. 217.

Now, suppose such a body to have acquired a slight displacement, with the result that it has taken up the position shown in fig. 217. The centre of gravity (G) remains unchanged, but the centre of buoyancy (B) has been removed to a point which does not lie vertically below the centre of gravity. Obviously there now exists a couple, the moment of which, W x, is the weight of the body (W) into the horizontal distance (x) between the two centres. The moment is a righting moment, and tends to bring the pontoon back to its original position.

Suppose, however, the pontoon to float on its narrower side or end as shown in fig. 218. In the upright condition the primary condition of equilibrium obtains as before. But when a slight displacement takes place (fig. 219), the moment (W x) called into existence is an overturning instead of a righting moment, and the pontoon has every tendency to capsize. We see, then, that a different state of things has been produced, and it becomes necessary, therefore, to investigate the relative positions of the centres of gravity and buoyancy a little more closely.

In each of the figures, let the vertical line drawn through the centre of gravity when the pontoon is in its initial position, and also that through the

centre of buoyancy in the displaced condition, be continued until they intersect at a point which we will designate M. The technical name for this point of intersection is the **metacentre.**[1]

It will be noticed that there is a very striking difference in the position of the metacentre in the two figures. In one case it lies above the centre of gravity of the pontoon; in the other case, it lies below it. The former arrangement produces a righting moment; the latter, an overturning moment.

The metacentre of a floating body has a variable position dependent upon (*a*) the shape of the body and (*b*) its centre of gravity, and also (*c*) upon the centre of buoyancy; but, from what has been pointed out, it follows, as a

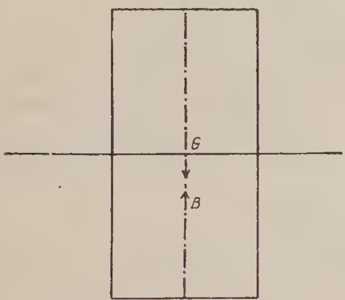


Fig. 218.

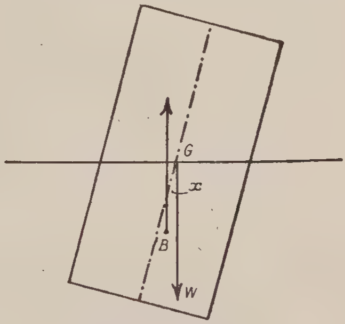


Fig. 219.

general rule, that a pontoon is stable or unstable according as the metacentre lies above or below the centre of gravity.

It would not be strictly correct to say that overturning would absolutely ensue in the latter event, as through the variation in the centre of buoyancy an intermediate position might be reached in which the conditions of equilibrium are satisfied.

It will be well, therefore, to go through the process of determining the complete range of position of the metacentre, and to construct a diagram showing all the changes in position of the centre of buoyancy corresponding to various degrees of inclination.

(I.) *Case of the unballasted pontoon immersed to half its depth.*—Let us take the case of a symmetrical pontoon of rectangular cross-section floating so as to have a moiety of its volume immersed. Fig. 220 represents such a case:

[1] Bouguer, who introduced the term a century and a half ago, employed it to designate a point in a ship's vertical axis above which the centre of gravity of the vessel might not be raised without producing an inclination in the axis. The metacentre must not be confused with any of a series of points on a curve distinguished by Bouguer as the *metacentric*. The metacentric may be defined as the locus of the intersections of successive verticals through adjacent centres of buoyancy, as a ship undergoes a series of slight inclinations. In other words, it is the evolute of the curve of buoyancy, or the locus of its centre of curvature.

G H N P is the pontoon, and R T the water surface level in the initial position, while F S, H P, and V X are other water-lines corresponding to changes of inclination in the pontoon.

In the initial position, the centre of buoyancy is at B_1 in the vertical line Q K passing through O.

Now, suppose the pontoon be acted upon so as to take up an inclination in which the water line is F O S. The immersed section becomes F H N S, and the centre of buoyancy which corresponds to this position lies somewhere to the right of B_1. We have to determine its position

On examination, we see that of the immersed area, a triangular wedge R F O, representing upward pressure both in amount and intensity, has been transferred to the other side of the axis K Q—viz., to T S O. Accordingly, by a principle of mechanics, the centre of gravity of the rectangle R H N T has been moved along a line parallel to the line joining the centre

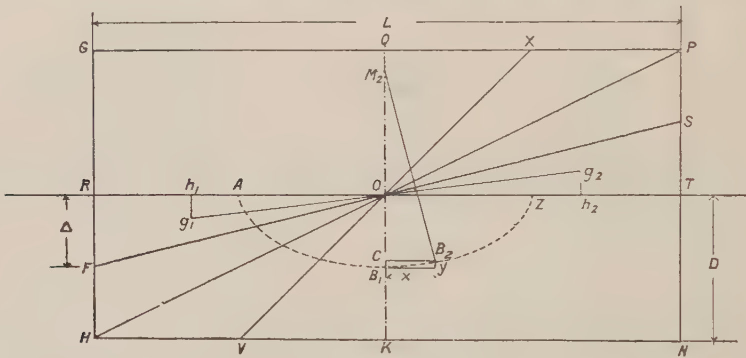


Fig. 220.

of gravity of the equal triangles R F O and T S O, a distance measured by that between the centres of gravity of the triangles, multiplied by the ratio of the area of one triangle to the area of the original rectangle. To put this in symbols, let g_1 be the centre of gravity of the triangle R F O, and g_2 the centre of gravity of the triangle S T O. Then, if B_2 be the position of the new centre of buoyancy, we must have

(1) The line B_1B_2 parallel to g_1g_2;

(2) The distance B_1B_2 equal to $g_1g_2 \times \dfrac{\text{area R F O}}{\text{area R H N T}}$.

Since the alteration in the position of B corresponds to the direction of g_1g_2, and, therefore, is partly an upward movement, we may conveniently find the locus by co-ordinates, with the initial position of B as origin. A line through B_1, parallel to R T or H N, will be the axis of x; the line K Q, the axis of y.

Draw g_1h_1 and g_2h_2 perpendicular to R T.

The abscissa of B_2 is proportionate to h_1h_2; its ordinate to $g_1h_1 + g_2h_2$.

Call O R, the semi-width of the pontoon, $\frac{L}{2}$; R H, the original depth of immersion, D; and R F, the extent of emergence, Δ.

Then, by the ordinary principles and operations of trigonometry and mechanics, we have

$$h_1h_2 = 2h_1O = \frac{2}{3}L.$$

Similarly,

$$h_1g_1 + h_2g_2 = \frac{2}{3}\Delta,$$

and the ratio

$$\frac{\text{area R F O}}{\text{area R H N T}} = \frac{\frac{1}{2}\text{F R . R O}}{2\text{R O . R H}} = \frac{\text{F R}}{4\text{R H}} = \frac{\Delta}{4\text{D}}.$$

Whence we obtain as values for the co-ordinates.

$$x = \frac{2}{3}L \cdot \frac{\Delta}{4D} = \frac{\Delta L}{6D}$$

and

$$y = \frac{2}{3}\Delta \cdot \frac{\Delta}{4D} = \frac{\Delta^2}{6D}.$$

The equation of the locus accordingly is

$$y = \frac{6D}{L^2}x^2.$$

It is at once apparent that, L and D being fixed quantities, y varies directly as x^2—that is, the locus of B is the curve of a parabola whose longitudinal axis is Q B_1, and vertex, B_1.

These equations for x and y hold good up to the point where Δ becomes equal to D. Giving Δ the value $\frac{D}{2}$, we obtain

$$x = \frac{L}{12};\ y = \frac{D}{24};$$

and, giving it the value D,

$$x = \frac{L}{6};\ y = \frac{D}{6}.$$

This brings us to the diagonal H O P, at which inclination the side G P commences to be immersed and the side G H is entirely out of water. It will simplify matters now if we regard the pontoon as undergoing disturbance from an initial position in which the vertical axis is R T, and the surface level

K Q, for that is the position towards which the pontoon is tending in the continuation of its revolution.

The calculations for the locus of B in reference to the new axis will equally give a parabolic curve, having its vertex at Z, where $O\,Z = \frac{L}{4}$.

Accordingly, the locus of B resolves itself into a curve consisting of four parabolic arcs touching at the diagonals, H P and G N, of the parallelogram. A moiety of the curve is traced in fig. 220.

Now, to find the metacentre corresponding to any assigned centre of buoyancy, it is only necessary to draw, perpendicular to the water surface, a line from the given point B_2 on the buoyancy curve until it intersects the axis Q K.

Its position may be located algebraically, thus—

$$\begin{aligned} B_1M_2 &= M_2C + C\,B_1 \\ &= M_2C + y. \end{aligned}$$

Now, the triangles $C\,B_2\,M_2$ and R F O are similar, their sides being respectively at right angles to one another.

Therefore

$$\begin{aligned} M_2\,C &= \frac{O\,R}{R\,F} \cdot C\,B_2 \\ &= \frac{L}{2\,\Delta}\,x, \end{aligned}$$

and

$$B_1\,M_2 = \frac{L}{2\,\Delta}\,x + y.$$

But

$$x = \frac{\Delta\,L}{6\,D} \text{ and } y = \frac{\Delta^2}{6\,D},$$

whence substituting

$$B_1\,M_2 = \frac{L^2}{12\,D} + \frac{\Delta^2}{6\,D}.$$

So when Δ is zero in the initial position, $B_1\,M_2 = \frac{L^2}{12\,D}$; and when $\Delta = R\,H$,

$$B_1\,M_2 = \frac{L^2}{12\,D} + \frac{D}{6}.$$

The range of position of the metacentre is accordingly within a length $\frac{D}{6}$ upon the axis Q K, and, similarly, within a length $\frac{L}{12}$ upon the axis R T.

A much simpler method, however, may be devised for obtaining the position of the metacentre by geometrical construction.

It will be observed that the equation of the parabola,

$$y = \frac{6\,D}{L^2}\,x^2,$$

is satisfied by the values $x = \frac{L}{2}$, $y = \frac{3\,D}{2}$; that is to say, the curve passes through the points G and P, which are the uppermost corners of the pontoon. Knowing also the vertex, B_1, of the curve, it is a very easy matter to construct, by any of the recognised methods, the parabolic arc G B_1 P,* the portion of which, $\alpha\,\beta$ lying between the semi-diagonals H O and O N, constitutes a part of the buoyancy curve (fig. 221).

Now, for any assigned water-line F S, take the chord $f\,s$ on this line which is cut off within the parabola, and bisect it at the point C. Draw C B_2 parallel

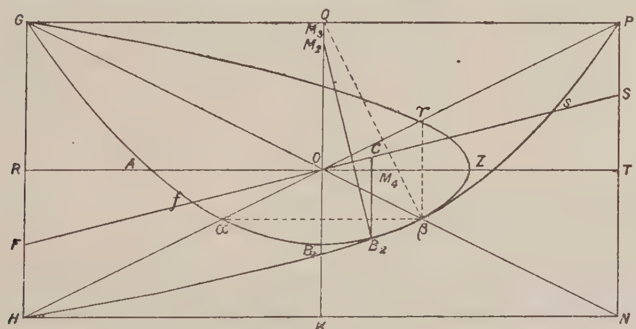


Fig. 221.

to the axis Q K, intersecting the curve at the point B_2. Then B_2 is the centre of buoyancy for the water-line F S. From B_2 draw $B_2\,M_2$ at right angles to F S so as to cut the axis Q K in M_2. M_2 is the metacentre under the same conditions.

The proof of the foregoing construction lies in the facts that the tangent at the extremity of a diameter of a parabola is parallel to the chords which are bisected by that diameter, and that the tangent to the curve of buoyancy at any point is parallel to the line of flotation corresponding to that point as centre of buoyancy.

Thus far, we have only dealt with centres of buoyancy lying within the section H O N; that is for water-lines ranging between G N and H P.

For water-lines between H P and N G, it is necessary to construct the parabola G Z H. This being done (fig. 221), the curve of buoyancy is defined from β to γ, and the corresponding positions of the metacentre may be determined as already explained. They will lie, however, not on the axis Q K, but on the axis R T, parallel to which the lines, from the water surface to the curve, must be drawn.

* *Vide* construction, fig. 228, p. 286.

It is interesting to consider the water-line H P. There are two parabolic chords appertaining to the position—viz., H γ and α P. The middle point of the former is α, and of the latter γ. Both the line $\alpha\,\beta$, drawn parallel to the axis R T of the parabola G Z H, and the line $\gamma\,\beta$, drawn parallel to the axis Q K of the parabola G B_1 P, meet at the point β, which is the limiting position of the centre of buoyancy for both parabolas. The line $\beta\,M_3$, perpendicular to H P, gives extreme positions for the metacentre on the respective axes: the upper limit M_3 on Q K, and the lower limit M_4 on R T. In fig. 221, owing to the particular ratio of L to D, the point M_3 coincides with the point Q.

The curve of buoyancy for the remaining moiety of the pontoon is simply a replica, upon the other side of the diagonal H P, of the curve $\alpha\,\beta\,\gamma$.

The parabolas G B_1 P and G Z H have three point contact—that is, the curves not only touch one another, but cross when continued.

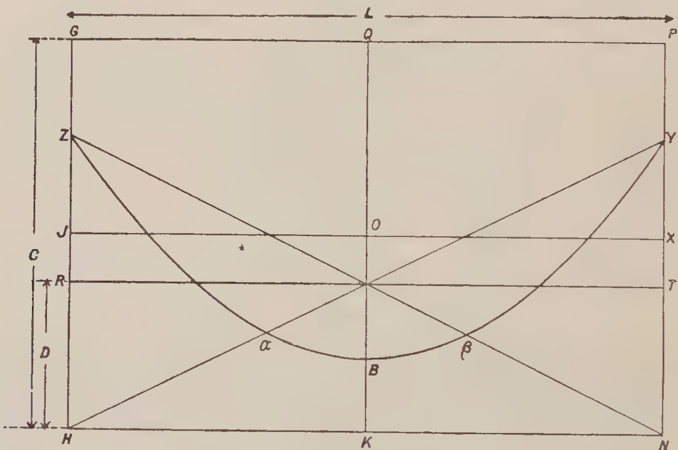


Fig. 222.

(II.) *Case of the unballasted pontoon immersed to any fraction of its depth.*—The foregoing is a special case in which the pontoon floats with exactly one-half of its bulk immersed.

From the principles enunciated, and by means of the methods which have been described, it is not difficult to determine the locus and draw the curve of buoyancy for the general case in which the pontoon floats with any proportion of its volume under water.

Let fig. 222 represent the conditions in question, the surface level of the water, R T, not being coincident with J **X**, the horizontal axis of symmetry of the pontoon.

In the initial position, the centre of buoyancy lies at the point B on the axis Q K, which is such that B K $= \dfrac{\text{R H}}{2} = \dfrac{\text{D}}{2}$. Taking B as origin and a

horizontal line through B as axis of x, we have (as explained in the preceding investigation) for all values of Δ between zero and D,

$$x = \frac{\Delta L}{6 D} \text{ and } y = \frac{\Delta^2}{6 D},$$

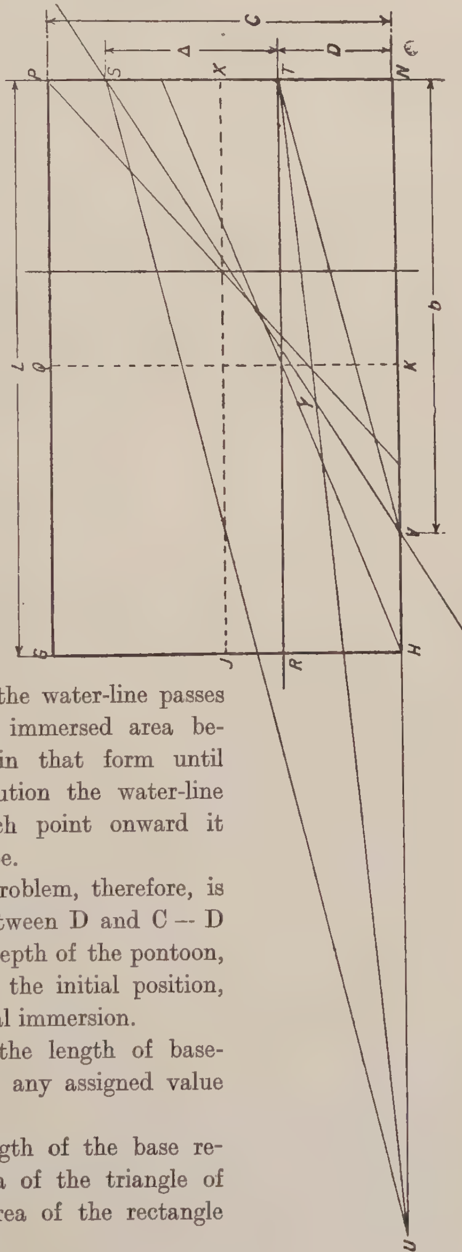


Fig. 223.

giving, as the equation of the curve,

$$x^2 = \frac{L^2}{6 D} y.$$

The parabola thus defined passes through the points Z and Y, where R Z = T Y = D, and constitutes the locus of the centre of buoyancy within the limits α and β situated on the diagonals of the parallelogram Z H N Y.

So long as the corner, H, of the pontoon remains under water, the immersed section is a quadrilateral. When the point H lies on the surface, the water-line passes through the point Y and the immersed area becomes triangular, remaining in that form until in course of continued revolution the water-line passes through P, from which point onward it resumes the quadrangular shape.

The second phase of the problem, therefore, is to deal with values of Δ between D and C − D (fig. 223), where C is the full depth of the pontoon, D the depth of immersion in the initial position, and Δ the subsequent additional immersion.

First, we must determine the length of baseline on H N corresponding to any assigned value of Δ, say T S.

Let b (fig. 223) be the length of the base required. Then, since the area of the triangle of immersion must equal the area of the rectangle R H N T, we have

$$\frac{b}{2}(D + \Delta) = L D; \quad \therefore\; b = \frac{2 L D}{\Delta + D};$$

or, we can find b geometrically thus :—In fig. 223 produce N H to U, so that U H = H N. Join S U. Through T draw T V parallel to S U, cutting N H in V. Join S V. Then S V is the desired water-line, and V N = b.

The proof of the construction is simple. The triangles S V U and S T U are equal, being on the same base and between the same parallels. Deduct the common portion S Y U, and add to each, in place of it, the trapezium V Y T N. Then the triangle S V N is equal to the triangle U T N, which is also equal to the rectangle R H N T, being on double the base, between the same parallels.

Reverting to the algebraical value of $b \left(= \frac{2\,L\,D}{\Delta + D}\right)$, and taking x and y as the co-ordinates of the centre of gravity of the triangle S N V, referred to the same axes as before intersecting at B, then

$$x = \frac{L}{2} - \frac{1}{3}\left(\frac{2\,L\,D}{\Delta + D}\right) = \frac{L}{6} \cdot \frac{3\,\Delta - D}{\Delta + D}$$

and
$$y = \frac{\Delta + D}{3} - \frac{D}{2} = \frac{2\,\Delta - D}{6},$$

whence we can obtain the equation for the locus, viz. :—

$$36\,x\,y + 18\,D\,x - 18\,L\,y - L\,D = 0,$$

which indicates an hyperbola.

Transfer the origin to x_1, y_1. Then

$$36(x + x_1)(y + y_1) + 18\,D\,(x + x_1) - 18\,L\,(y + y_1) - L\,D = 0.$$

If the coefficients of x and y in this expression be made equal to zero, the values of x_1 and y_1, corresponding thereto, will give the centre of the curve.

Thus
$$36\,y_1 + 18\,D = 0,$$
and
$$36\,x_1 - 18\,L = 0.$$

Therefore,
$$y_1 = -\frac{D}{2},$$
and
$$x_1 = \frac{L}{2}.$$

Accordingly, the point N is the centre.

Referring the equation to N as origin, with N H and N P as axes, it becomes

$$36\left(x + \frac{L}{2}\right)\left(y - \frac{D}{2}\right) + 18\,D\left(x + \frac{L}{2}\right) - 18\,L\left(y - \frac{D}{2}\right) - L\,D = 0;$$

whence
$$x\,y + \frac{2}{9}\,L\,D = 0.$$

Therefore, N H and N P are the asymptotes of the rectangular hyperbola constituting the curve. Further, since

$$x\,y = -\frac{2}{9}\,\mathrm{L\,D},$$

we see that the locus lies on the conjugate hyperbola.

If the transverse axis of a rectangular hyperbola be $2\,a$, then

$$4\,x\,y = -\,2\,a^2$$

is the equation of the conjugate hyperbola.

This may be written

$$x\,y = -\frac{a^2}{2}.$$

Comparing it with the preceding value of $x\,y$, we see that

$$\frac{a^2}{2} = \frac{2}{9}\,\mathrm{L\,D};$$

that is, $$a = \frac{2}{3}\sqrt{\mathrm{L\,D}}.$$

Turning again to the general equation,

$$36\,x\,y + 18\,\mathrm{D}\,x - 18\,\mathrm{L}\,y - \mathrm{L\,D} = 0,$$

it will be observed that the curve passes through the point $\frac{\mathrm{L}}{6}, \frac{\mathrm{D}}{6}$.

Further, taking the equation of the parabola in the first phase—viz.,

$$x^2 = \frac{\mathrm{L}^2}{6\,\mathrm{D}}\,y,$$

where the two curves intersect, we get

$$36\,.\,\frac{6\,\mathrm{D}}{\mathrm{L}^2}\,x^3 + 18\,\mathrm{D}\,x - 18\,\mathrm{L}\,\frac{6\,\mathrm{D}}{\mathrm{L}^2}\,x^2 - \mathrm{L\,D} = 0,$$

or $$(6\,x - \mathrm{L})^3 = 0.$$

Therefore, the parabola and hyperbola have three point contact at $\frac{\mathrm{L}}{6}, \frac{\mathrm{D}}{6}$.

The curve may now be traced from the foregoing data.[1] This being done (fig. 224), it remains to find the centre of buoyancy corresponding to any possible water-line within the limits already specified. Take V S as such a line and, as in the case of the parabola, bisect that portion of it, $f\,s$, which forms a chord of the hyperbola. Join this middle point C to N the centre

[1] The method is shown in fig. 227.

of the curve. Where the line C N intersects the curve is the centre of buoyancy for the position, and a line drawn therefrom at right angles to the water-line will cut the original vertical axis in the metacentre. Thus B

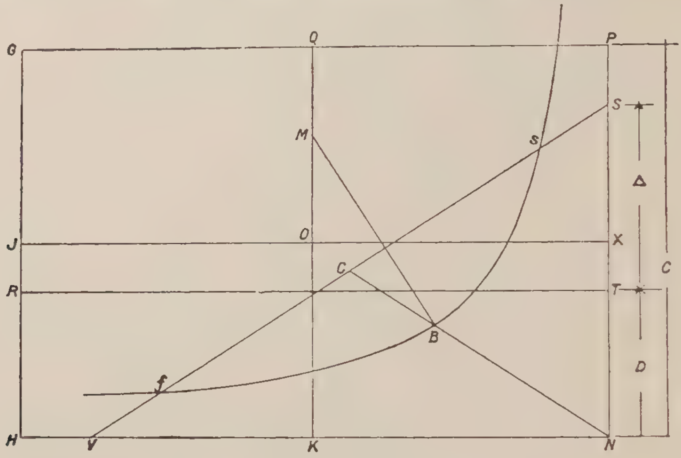


Fig. 224.

and M are the centre of buoyancy and the metacentre respectively, for the water-line V S.

The last phase of the problem is the same as the first. When the pontoon is immersed as with its axis J O X (fig. 225) vertical in the initial position,

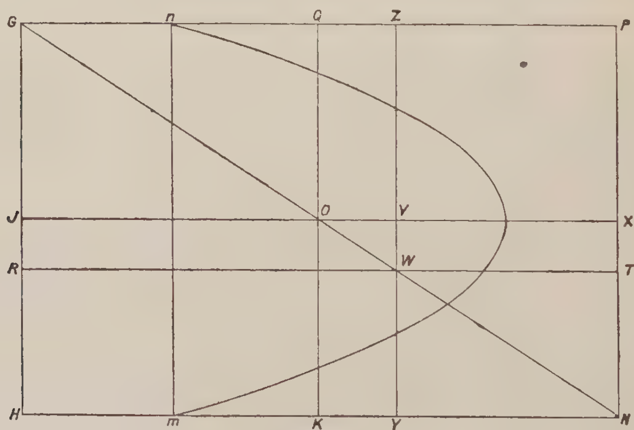


Fig. 225.

we have a similar set of conditions to those in which the axis Q O K was initially vertical.

To find the depth (D_1) immersed, we have

$$D_1 C = L D \text{ or } D_1 = \frac{L}{C} D,$$

or, geometrically, thus (fig. 225):—Through the point W, in which the diagonal N G intersects the primary water-line R T, draw the line Y Z. Then Y Z is the new water-line, and V X $= D_1$.

It follows, exactly as before, that the curve of buoyancy for the latest condition of things is a parabola, the vertex of which is in J X at a distance $\frac{D_1}{2}$ from the point X, and which passes through the points m and n in the lines H N and G P, where N m = P n $= 2\,D_1$.

We have now traced the curve of buoyancy through rather more than half of its path, and the remaining portion lies symmetrically about the same axes, so that it is quite easy to draw the entire curve. A moiety is shown in fig. 226. In the left-hand quadrant the parabolic arc extends from A

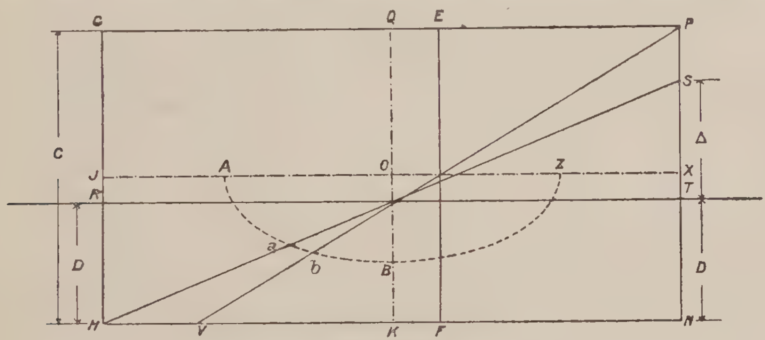


Fig. 226.

to a, to be succeeded by a hyperbolic curve from a to b. From b to B the curve is parabolic once more.

Hence, the complete locus of the centre of buoyancy is a curve made up of four parabolas and four hyperbolas.

When $D = \frac{C}{2}$, the hyperbolic portion vanishes and we get the special case of a pontoon immersed to one-half its depth, the investigation of which occupied our attention at the outset.

Method of Tracing Parabolic and Hyperbolic Curves—*Parabola.*—Divide the semi-base and the height into an equal number of parts, as shown in fig. 228. Draw lines parallel to the axis through each of the points on the base, and join the points on the sides to the vertex. The intersections give points on the curve.

Hyperbola (see fig. 227).—The curve required is the rectangular hyperbola, and the distance a from the centre H to the vertex B is given. Draw the axis H X bisecting the angle G H K. With centre H and radius a, describe the quadrant A B C. Join A C. From C draw C E at right angles to H K, cutting the axis in the point E, which is the focus. Through D draw a horizontal line of indefinite length.

Take any number of parallel lines intersecting the axis at right angles. Measure along the horizontal line through D the distance from D to each point of intersection. With these distances as radii respectively, and the focus as centre, mark corresponding intersections on the parallel lines, on both sides of the axis. These intersections are points on the curve.

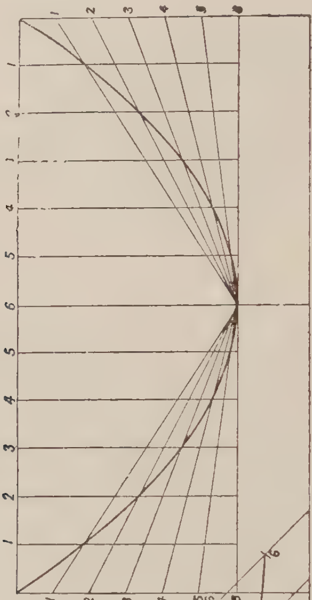


Fig. 228.—Parabola.

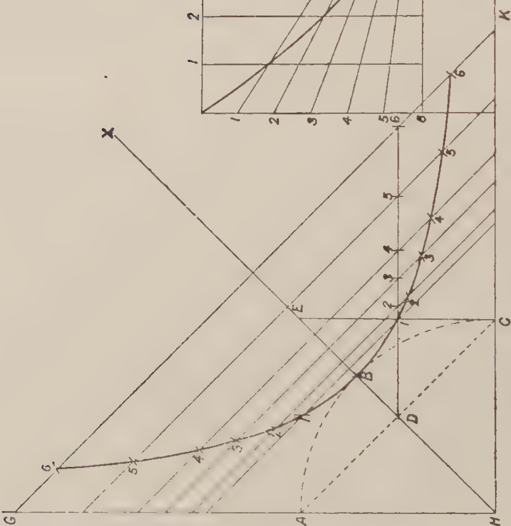


Fig. 227.—Hyperbola.

Note.—In the investigation on p. 283, the value of a was found to be $\frac{2}{3}\sqrt{\text{L D}}$. To obtain the value graphically, take a line (fig. 229) whose length is L + D, and divide it into two parts equal to L and D respectively. Upon the line describe the arc of a semi-circle, and from the point of section draw

a vertical line to meet the semi-circle. This line has a length $\sqrt{LD}$, and two-thirds of it will give the value a.

(III.) *Case of the ballasted pontoon.*—We have so far only considered the pontoon as an empty box—independent and self-contained. We have now to regard it in its working aspect. It is intended to carry a load, and for purposes of insertion and withdrawal from position it is oftentimes ballasted with water. The object of admitting water to the interior of the pontoon is to enable it to be lowered temporarily to a deeper draught, from which it can be raised again by pumping out the water.

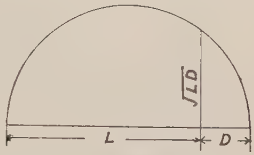


Fig. 229.

We will deal first with the matter of ballasting.

When the pontoon is a single box, without compartments, the introduction of water diminishes its stability, as will be evident from the diagram. The fluid, instead of remaining equally distributed under disturbance, immediately flows to the deeper side and there assists the overturning moment by its impetus, or, at least, impedes the righting effort.

The buoyancy, moreover, of the pontoon is reduced by the occupation of internal space.

The drawback of shifting ballast may be to some extent mitigated by subdividing the pontoon into compartments. When disturbed, the distribution of the water becomes less markedly unequal, as is evident from figs. 230 and 231. The greater the number of compartments the more uniform

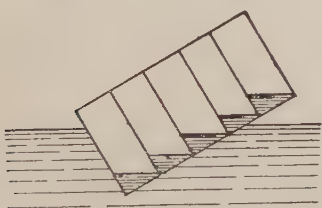


Fig. 230.

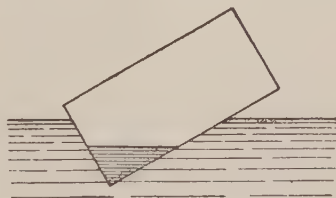


Fig. 231.

the ballasting will become; but, of course, there are limits of economy and accessibility to be considered.

Before investigating the case of the ballasted pontoon, it is desirable to reconsider the meaning to be attached to the term "centre of buoyancy" which we have hitherto found an important and essential feature in the statics of the unballasted pontoon.

After displacement through an angle, the forces tending to restore a ballasted pontoon to its original position, or to move it further from that position, are:—

(*a*) The weight of the displaced water, acting upwards through its centre of gravity.

(*b*) The weight of the ballast water acting downwards through its centre of gravity.

(*c*) The weight of the pontoon acting downwards through its centre of gravity, which, in the case of a rectangular pontoon, is the centre of that figure.

The resultant of the first two forces is a force equal in magnitude to the third force, but opposite in direction, and acting through a point which may be described as the centroid of the buoyancy area.

This resultant and the force (*c*) form the righting couple, and henceforth the term centre of buoyancy must be understood in the sense of **centroid of the buoyancy area** which it really is. To avoid misconception on the point, it will perhaps be as well to adopt the expression "centroid of buoyancy."

Resuming the investigation of the stability of the pontoon, it is, in the first place, desirable to consider the alterations in position of the centroid of buoyancy in a pontoon which is subdivided into compartments. Let

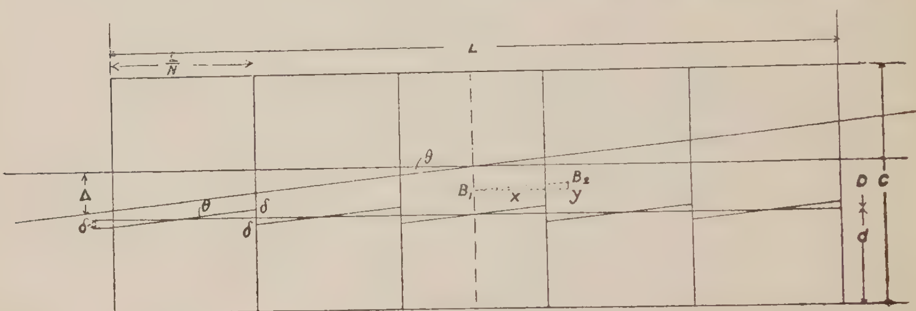


Fig. 232.

fig. 232 represent such an arrangement, the number of compartments in this case being five.

Under a slight displacement, the pontoon takes the position shown with reference to the inclined line in fig. 232. The buoyancy area has been changed from a rectangle into a series of parallelograms. A little inspection will, moreover, show that the portion of buoyancy area deducted from the extreme left-hand compartment has been added to the compartment on the extreme right, and that the adjoining compartments of each have likewise experienced a similar transfer of area.

The most obvious way, therefore, of determining the new position of the centroid of buoyancy is to sum up the products of the individual areas transferred, into the distances of their respective transferences, and divide by the whole buoyancy area. This will give us the proportionate transference of the centroid of buoyancy of the original rectangle.

Also, it will be well to proceed by means of the horizontal and vertical components, as before, which yield the co-ordinates of the locus.

Let D (fig. 232) be the depth of buoyancy as originally immersed, and Δ the extent of emergence or submergence of either side under an angular displacement θ. The emergence or submergence in any compartment of N compartments we will call δ. If d be the original depth of water inside the pontoon, then $D + d$ equal the depth of the pontoon.

We have

$$\Delta = \frac{L}{2} \tan \theta.$$

Also from similar triangles,

$$\delta = \frac{L}{2\,N} \tan \theta.$$

Therefore,

$$\delta = \frac{\Delta}{N}.$$

The area of buoyant section transferred from the extreme left-hand to the extreme right-hand compartment (1st to 5th) is

$$\left(\Delta - \frac{\Delta}{N}\right)\frac{L}{N},$$

and the horizontal distance between the centres of gravity of the two compartments is

$$\frac{N-1}{N}\,L.$$

The product of these two is

$$\left(\Delta - \frac{\Delta}{N}\right) \cdot \frac{L}{N} \cdot \left(\frac{N-1}{N}\right) L,$$

which, for the purpose of forming a series for summation, may be written

$$\Delta\left(\frac{N-1}{N}\right) \cdot \frac{L}{N} \cdot \left(\frac{N-1}{N}\right) L.$$

The similar product in the case of the 2nd and 4th compartments is

$$\Delta\left(\frac{N-3}{N}\right) \cdot \frac{L}{N} \cdot \left(\frac{N-3}{N}\right) L,$$

and, in the event of there being additional compartments, we could write as the next term,

$$\Delta\left(\frac{N-5}{N}\right) \cdot \frac{L}{N} \cdot \left(\frac{N-5}{N}\right) L.$$

If N be odd, the displacement for the middle compartment is zero.

Accordingly, we have the following series to summate :—

$$\frac{\Delta L^2}{N^3}\left\{(N-1)^2+(N-3)^2+(N-5)^2+\ldots\ldots\right\};$$

which comes to

$$\frac{\Delta L^2}{N^3}\cdot\frac{N(N^2-1)}{6}=\frac{\Delta L^2}{6}\left(1-\frac{1}{N^2}\right).$$

The area of the original rectangle is L D, and, dividing the preceding expression by it, we obtain the following value for the horizontal component of transference :—

$$x=\frac{\Delta L}{6 D}\left(1-\frac{1}{N^2}\right).$$

Similarly, it can be shown that

$$y=\frac{\Delta^2}{6 D}\left(1-\frac{1}{N^2}\right).$$

Comparing these equations with the values on p. 277 previously given for the ordinates in the case of the undivided and unballasted pontoon, we see that they differ only by the factor $\left(1-\frac{1}{N^2}\right)$, which is a constant for any assigned case. Therefore, so long as the displacement is confined within limits such that the bottom of the pontoon is not exposed either inside or out, nor the upper corners immersed, the curve of buoyancy is parabolic as before.

The metacentric height, measured above the primary centroid of buoyancy, is

$$\left(\frac{L^2}{12 D}+\frac{\Delta^2}{6 D}\right)\left(1-\frac{1}{N^2}\right);$$

or, measuring from the centre of depth O of the pontoon,

$$\left(\frac{L^2}{12 D}+\frac{\Delta^2}{6 D}\right)\left(1-\frac{1}{N^2}\right)-\left[\frac{C}{2}-\left(d+\frac{D}{2}\right)\right].$$

When in the upright position, $\Delta=0$, and the latter expression reduces to

$$\frac{L^2}{12 D}\left(1-\frac{1}{N^2}\right)-\left[\frac{C}{2}-\left(d+\frac{D}{2}\right)\right]$$

from which it is clear that with an increase in the number of compartments, the metacentric height increases, and, therefore, the greater the number of the compartments the greater the stability.

When $N=1$, the metacentric height is

$$-\left[\frac{C}{2}-\left(d+\frac{D}{2}\right)\right];$$

and as the metacentric height (measured, as assumed above, from the centre of depth) is proportional to the moment of the righting couple, the pontoon is in stable, neutral, or unstable equilibrium, according as

$$d + \frac{D}{2} \gtreqless \frac{C}{2}.$$

We turn now to the question of load, which only concerns the pontoon in its empty condition. When in position a pontoon carries kelsons, or main girders, which, in turn, receive the load of deck-beams and stringers. These imposed loads, beyond raising the centre of gravity, do not affect the external conditions of equilibrium; their immediate interest is in regard to the conditions of internal equilibrium.

Fig. 233.

Fig. 234.

The problem of the internal stresses to which pontoons are subjected is not one which need cause any difficulty in the way of solution, the same methods of procedure being applicable as in dealing with ordinary beams. There is but one difference, albeit a striking one, between the two cases; but the effect of this is not nearly so embarrassing as might at first sight appear. In a beam the upward reaction is concentrated at isolated points of support; in a pontoon the reaction is distributed over the whole of the immersed area. A very simple expedient serves, however, to put the two cases on an equal footing.

Take fig. 233, representing a beam uniformly loaded and supported beneath at any two points. Now, invert the diagram, as in fig. 234, and we have the case of a floating pontoon carrying two concentrated loads. Obviously, the same diagrams of shearing stress and bending moment will serve in both cases, and there will be no difficulty in proceeding by this method in most cases. Even when the pontoon does not float upon an even keel, a measure of the exact distribution of the upward force is given by the area of the buoyant section (fig. 235). Figs. 237 to 239 are consequently typical bending moment diagrams.

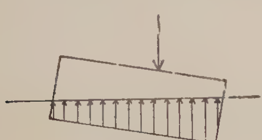


Fig. 235.

Fig. 236.

When a pontoon supports three or more concentrated loads, the conditions become identical with what is known as the case of the continuous beam, but,

in this instance, one of the chief obstacles to a ready determination of the problem is removed, in that the exact amount and dispositions of all the external forces are known.

Such being the conditions, it is unnecessary to pursue the matter further,

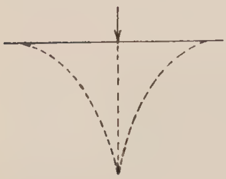


Fig. 237.

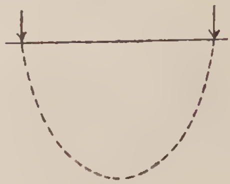


Fig. 238.

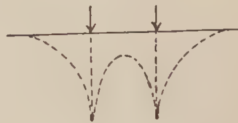


Fig. 239.

as it simply resolves itself into an application of the ordinary principles of mechanics to structural stresses in general, and lies, therefore, outside the special province of this volume.

Liverpool Floating Landing-stage.—The Liverpool stage, of which illustrations are given in figs. 240 to 242, showing the arrangement of the pontoons and decking, is 2,478 feet long by 80 feet wide. It has eight bridges connecting it with the shore, and, in addition, a floating bridge 550 feet long by 35 feet wide, by means of which an easy incline for goods traffic is maintained at all states of the tide, which, during springs, has a maximum range of about 30 feet. The constructive arrangement of the stage is evident from the figures.

Figs. 240 and 241.—Liverpool Floating Landing-stage.

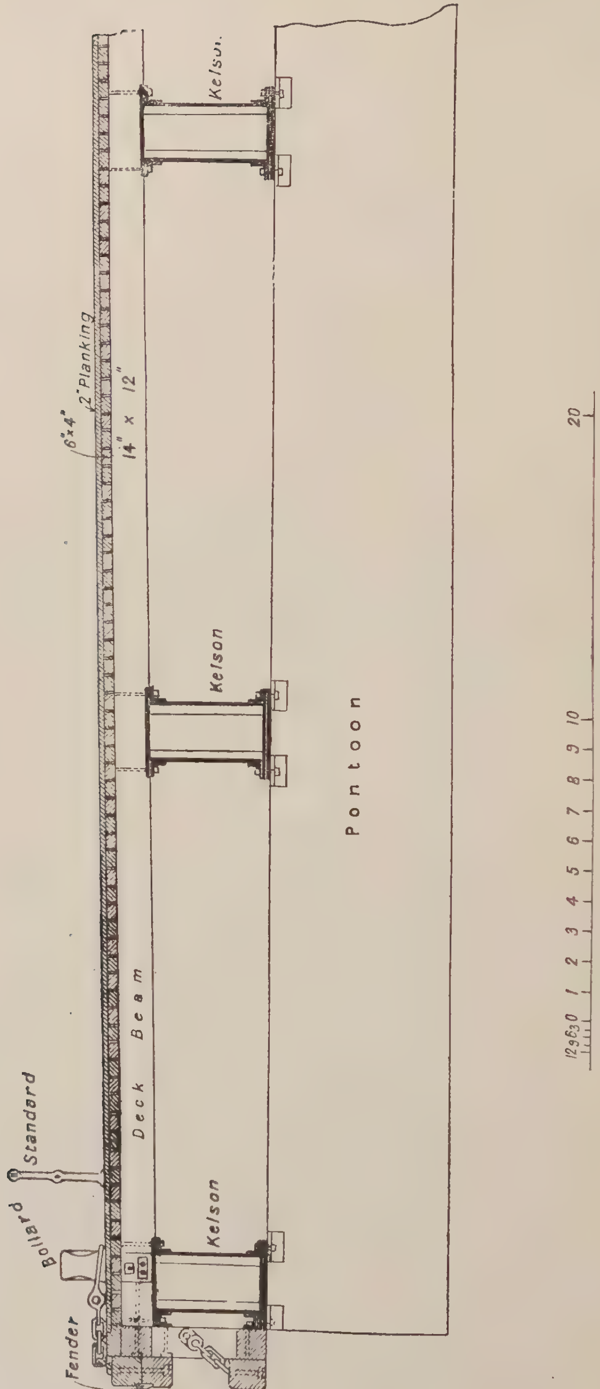


Fig. 242.—Liverpool Floating Landing-stage—Part transverse section

CHAPTER X.

ENTRANCE CHANNELS.

Variation in Conditions—Features of a Tidal Régime—Blind Channels—Variable Channels—Fixed Channels—Accretion and Reclamation—Navigable Routes—Bars and their Origin—Training Works for Channels—Groynes—Walls—Fascines—Wave Traps—Height and Extent of Training-walls—Dredging Appliances—Mechanical Eroders—Rock-cutting—Suction Dredgers—Sluicing—Instances of Channel Regulation Works at the mouth of the River Weser, Germany; at Tampico Harbour, Mexico; at Westport Harbour, New Zealand; at the mouth of the Richmond River, New South Wales; and at the Ports of Ostend, Belgium, and of New York, U.S.A.

Variable Conditions of Entrance Channels.—The regulation, and, where necessary, the rectification of entrance channels, are matters of extreme moment to all ports which are situated otherwise than upon the open seaboard, and especially do they call for attention in connection with harbours located within an estuary or upon the banks of a river within tidal range. In the case of ports lying either upon a river flowing into a tideless sea, or upon

Fig. 243.—Type of Normal River Flow. Main bed of stream shown by dark lines; shoaling shown by tinting.

a tidal river above the limits of tidal access, the agencies determining the form and direction of the river bed are, comparatively speaking, fixed and constant. The stream follows certain well-defined laws which, if not thoroughly understood, are at least clearly enunciated and expressed. It is a matter of general knowledge that in pursuing its sinuous course to the sea, the current of a river, as is indicated in fig. 243, impinges alternately against each bank, scouring the concave side of a bend and being thence diverted to a similar concavity on the opposite side at the next deflection of the river-bed: all this being in accordance with the principles of centrifugal force and action. The tendency, therefore, is for the navigable channel to form and maintain itself along the line of current, and there are few or no conflicting agencies to interfere with or modify this tendency.

Features of a Tidal Régime.—Within the limits of tidal influence, however, the conditions are of a totally different character, and the dispositions arising therefrom become much more intricate and complex. The current does not always flow in a seaward direction. For a very considerable portion of the time it is completely reversed, and a large bulk of sea-water is directed inland with a velocity which is sufficient to overcome the fresh-water discharge, and to raise the level of the surface for some distance. The main body of the incoming current, moreover, does not by any means necessarily pursue the same course as that of the downward stream; neither does it essentially confine itself, more especially near the river mouth, to one definite bed or channel. The influences at work upon littoral currents at the entrances of large coastal inlets are manifold and powerful. Gales

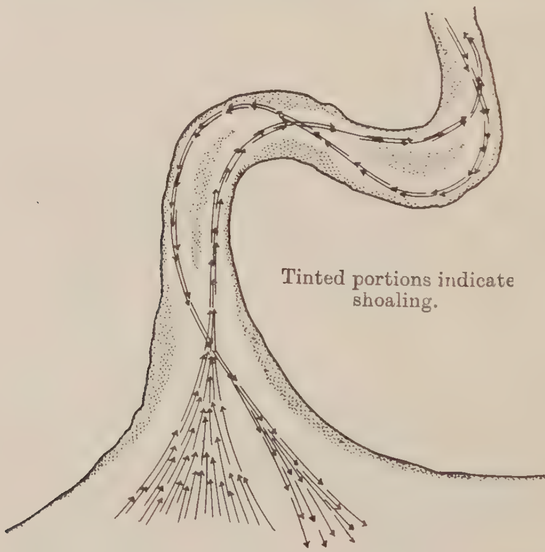


Fig. 244.—Diverse Courses of Inward and Outward Currents in Rivers.

and storms arise at irregular intervals from varying points of the compass, and the pressure exerted thereby is inevitably felt by the tidal flow which is accelerated or retarded, augmented or reduced, to an appreciable extent. The action of the wind, moreover, produces certain changes of direction. Hence, the course of the incoming tide, though generally established, is subject to some mutation, and the volume of its flow fluctuates very considerably, not only in consequence of irregular meteorological phenomena, but also in conformity with the natural cycle of springs and neaps.

In the tidal region, therefore, there are two conflicting agencies; first, the downward stream, with its relatively uniform flow and its tendency to establish a definite bed, and secondly, the tidal current with exactly opposite characteristics. The resulting feature of the tidal estuary is accordingly

Figs. 245 to 247.—Conformations of part of River Hooghly at different dates.

unstable channels amid shallows and sandbanks. Through the latter, the river, taking the line of least resistance as it presents itself at the moment, ploughs its course to the sea in routes which the succeeding tides break through, destroying them in succession as they are formed, before they have time to become confirmed. In the natural order of things, the fluvial current has acquired a sinuous or spiral motion, deflecting it from side to side, while the flood-tide sets inwards in a straight line, curbed by none of the influences which control the river. The tendency of each is to obliterate the traces of the other where they diverge, and to accentuate the common bed where they coincide. At certain points, the river follows a course along one bank, while the main tidal stream favours that opposite, with the result that there are intermediate zones of slack water conducing naturally to the formation of shoals. A typical example of this, if it be necessary to select one, is furnished by the redoubtable "James and Mary" shoal, which constitutes the most dangerous obstacle to navigation in the River Hooghly, and which has the evil reputation of being one of the most fruitful sources of shipwreck and disaster of any river in the world. The late Professor Vernon-Harcourt, who made a special and exhaustive study of the River Hooghly in 1901, thus describes the circumstances of the formation of the shoal[1] :—

"The descending current of the freshets in the rainy season scours out a deep channel alongside the concave left bank, and, diverging only slightly from this bank on passing Nurpur Point (figs. 245 to 247), it goes straight across the river into the very deep channel along the concave right bank below the bend a little beyond Gewankhali; whilst the deep flood-tide channel along the right bank, from Mornington Point towards Shipgunj Point, becomes more or less silted up during the prevalence of the freshets. On the other hand, during the dry season the flood-tide channel, or Western Gut, along the right bank, in the lower part of the reach, opens out again, till the scouring energy of the flood-tide current is dissipated to some extent on approaching Shipgunj Point, where it spreads out and passes across to the left bank between Ninan and Fulta Point. At the same time, the Eastern Gut near the left bank, which depends on the ebb-tide almost entirely for its maintenance throughout the dry season, is reduced in depth; and a bar is formed between the ebb-tide channel near Hooghly Point and the deep channel at right angles to it in front of Gewankhali, by the conflicting action of the flood-tide running up this latter channel, thereby joining the James and Mary Shoal to the Hooghly Sand below."

Blind Channels.—This conflict of routes, resulting in the temporary predominance of one or other, is also responsible for the formation of what may be termed "blind channels" or cul-de-sacs, such as will be noticed in greater or less prominence on the charts of all important estuaries. These are deep depressions in the river-bed extending for some distance without any apparent outlet, terminating simply in a ridge or stopped end. The

[1] Vernon-Harcourt on the River Hooghly, *Min. Proc. Inst. C.E.*, vol. clx.

Sloyne in the River Mersey is a notable example, as also the Bog Hole off Southport in the estuary of the Ribble, Mostyn Deep at the entrance of the Dee, and the Great Nore Channel at the mouth of the Thames. These and many other instances are undoubtedly due to the antagonistic tendencies of the upward and downward streams.

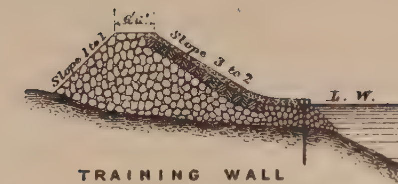
Variable Channels.—The roving disposition of channels in sandy estuaries is manifestly the cause of much waste of physical power. The energy possessed by the stream in virtue of its momentum, which might profitably be expended in maintaining a deep clear channel and in removing or preventing any obstruction of the nature of a bar, is dissipated in the effort of eroding and displacing huge volumes of sand. It is observable in the river Mersey, for instance, that the estuarine channel rarely occupies the same position for a week consecutively. Between Hale Head and Garston, where the estuary is three miles wide, the channel has been diverted within a period of twelve months across the entire width from the Cheshire side to the Lancashire side, and *vice versa.* Generally the changes are found to be coincident with upland floods, which bring a considerable accession of water; but so trifling are the initiatory causes sometimes, that barges grounding against a side of the existing channel have been known to produce a most marked deflection. The agitation arising from the process of erosion must inevitably cause a considerable quantity of sand to remain in suspension and to be transported to the mouth of the river, where its deposition in more tranquil waters is only a matter of time.

Fixed Channels.—A constant channel, on the other hand, where such can be assured, has all the advantages attaching to fixity and stability. It entails no frequent surveys with alterations of buoys and lights; it does its own maintenance work, and it acts generally on the lines of an ideal stream.

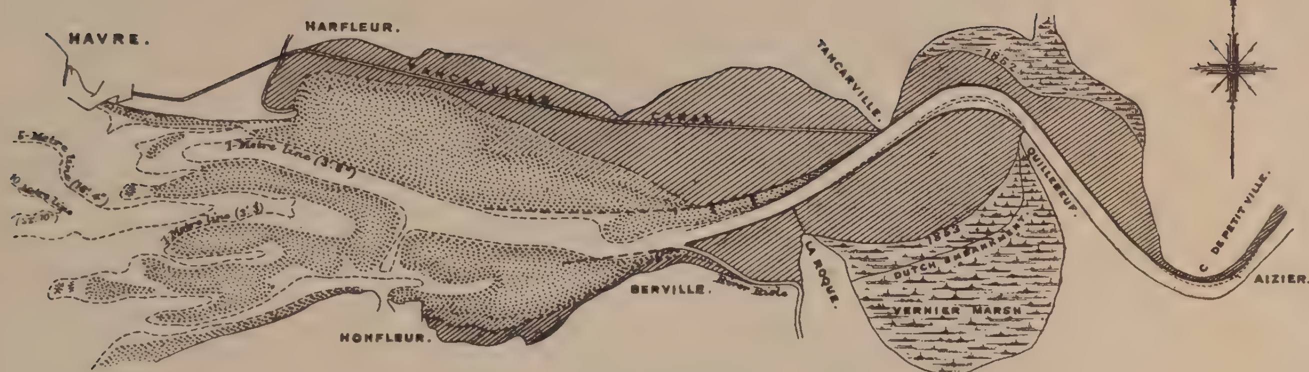
One forcibly impressive claim which has been put forward on behalf of a roving channel is, that by its constant change of course it deters the estuary from silting up in any part. This contention is one which has no little weight, because, with the reduction in capacity of a tidal basin or compartment, there is a corresponding reduction in the quantity of flood-water admitted, and a loss of scouring effect on the subsequent ebb. The confinement of a channel within restricted boundaries inevitably leads, in the case of water heavily charged with silt, to accretion in the adjacent submerged area. In other words, channel-training is a preliminary to land reclamation, and land reclamation is the general outcome of channel-training. Land reclamation is not an unmixed benefit; it may be attended by serious consequences to ports situated between the locality of reclamation and the sea, and it may entail other physical disabilities not altogether easy to foresee. Considerable discretion is, therefore, required both in planning and in carrying out undertakings embodying any such scheme.

Fixed *v.* Variable Channels.—Taking the question of river-training, how-

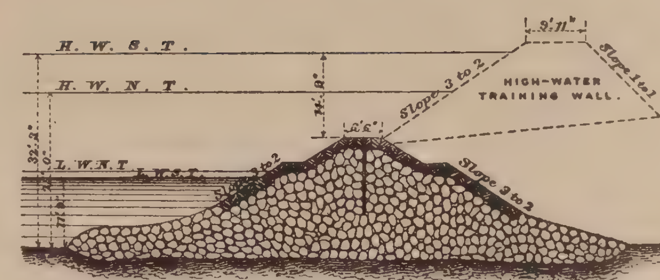
To face p. 299.]



TRAINING WALL AS RECONSTRUCTED.



TRAINING WORKS, SEINE ESTUARY, 1898: PLAN.



SOUTH TRAINING WALL EXTENSION.

Figs. 248 to 250.

NOTE.—The training walls have been carried to the full extent of the dotted lines.

ever, as a whole, on its intrinsic merits, it seems to turn on the point of relative advantages—whether, in fact, it is preferable to have a deep, narrow, well-defined, constant channel, with adequate energy for its own maintenance, but with none utilisable for counteracting any silting tendencies elsewhere, or, on the other hand, to have a shifting channel with a more sluggish flow, sluicing a large expanse of sand so as to keep it from consolidating in any part, and so affording a broad waterway of greater sectional area, but of inferior depth, and subject to all the inconveniences of a shallow bar. The first undoubtedly represents the ideal condition, but, as indicated above, there are practical and circumstantial grounds in some cases constituting a preponderating argument in favour of the latter.

Accretion.—Although it is oftentimes assumed that accretion is the inevitable consequence of confining a channel within narrow limits, yet such an assumption is not legitimate on all occasions. Accretion can only arise from the deposition of suspended sediment, and this sediment can only be forthcoming from a supply in excess of that which the outgoing stream can carry. Now, there is nothing to show that any additional detritus is forthcoming from the upper reaches of a regulated river. But even supposing that there be an augmentation, the increased velocity of the stream renders it capable of transporting a larger percentage of solid matter than before. Evidently, therefore, any deposition which takes place is hardly attributable to detritus brought down by the upland waters.

The more likely and, as a matter of fact, the only possible source of accretion, is a tidal flow laden with the harvest of coast erosion. The flood-tide, entering estuaries on a sandy coast, is almost universally heavily charged with mud and fine particles which have every tendency to deposit themselves at the period of slack water, unless the down stream be so directed as to bear upon the area of settlement, and this cannot be the case with a channel limited to one part of it.

The River Seine constitutes a typical illustration of the effects of training a channel through an estuary. Sixty years ago the outlet exhibited all the usual vagaries of estuarine channels in regard to alteration in position and irregularity of depth. From that time regulation works have been in hand, and the channel is now clearly defined from Rouen to some distance beyond Berville (fig. 248). At the outset, the probable results were entirely miscalculated; little or no consideration seems to have been taken of the question of silting, or rather, its potentialities were so under-estimated as to be deemed negligible. It was not long, however, before the consequences began to make themselves felt. Huge volumes of alluvium settled in the external vicinity of the training-walls, and the quantity increased rapidly as the capacity of the estuary to receive tidal water was diminished. Land reclamation followed as a natural sequel. But these processes, though beneficial in some respects, and by no means disadvantageous to the port of Rouen situated 74 miles up the river, became seriously prejudicial to

the port of Havre at its mouth. The entrance channels of this latter port began to shoal, sandbanks formed in the approaches, and Havre, as a port, was threatened with extinction. The training-works were arrested for a time. The gain to Rouen had been undoubtedly great; a serviceable channel was promoted and assured, so that, whereas formerly vessels of between 100 and 200 tons navigated the distance from the sea with difficulty, vessels of ten times that tonnage now effected the journey with ease. Moreover, the gain of land had appreciable advantages from a national point of view. Still, it was manifestly mistaken policy to consider that these benefits outweighed a depreciation in the prosperity of the port of Havre.

The difficulty was met by providing Havre with a sheltered deep-water approach direct from the open sea, entirely beyond the influence of accretion in the estuary of the Seine. With this step, involving the construction of two breakwaters of considerable extent, inclosing a new harbour and the formation of an entrance facing south-west, and outside the estuary altogether, freedom has been gained for prosecuting the training-works of the Seine, and these seem destined to be continued to the river's mouth.

Navigable Routes.—It must be pointed out, from a navigational point of view, that the vagaries of a shifting channel do not always entail an entire change of route for shipping. Deep gullies and guts may be excavated on the site of former shoals, and adjacent gullies may be silted up; but vessels entering and leaving a port do not necessarily follow the line of greatest depth. Such a line may, in fact, be associated with the blind channels already alluded to. A navigable channel, as a rule, consists of a series of deeps separated by intervening ridges or shoals, and the serviceability of the channel is governed by the depths of the latter. When any one of the ridges becomes unduly high for the draught of passing vessels, then, in the absence of remedial measures, it becomes necessary to lay down another route; but so long as the depth of water is adequate, this step need not be taken.

Bars.

The rectification and improvement of harbour approaches involves not only the training of channels, but in many cases also the removal of a bar—in part, at least.

A **bar** is a ridge or narrow plateau, or even a series of several ridges or plateaus, lying across the entrance to a river or coastal inlet, and rising up above the general level of the sea or river floor in its immediate neighbourhood, on both sides of it. When the altitude of the bar is sufficiently great to reduce the depth of water over its summit to an extent exceeding the limits imposed by the requirements of vessels using the entrance, it becomes an obstruction to navigation, and, in any case, it acts as an impediment to the development of the port or ports to which it is the threshold, and detracts from the navigable possibilities of the inlet.

Bars are to be found mainly in connection with tidal rivers; less often in connection with non-tidal rivers. On the other hand, some tidal rivers and many non-tidal rivers possess channels which, while they may becumbered and rendered tortuous by shoals, are entirely unobstructed by bars. The Mersey, the Dee, and the Rhone, for example, have bars of a very pronounced and indubitable character. The Thames, the Humber, and the Severn have channels which enter the sea without any marked obstruction at all of this nature.

The **origin of bars** has been the subject of some controversy. It was formerly pretty generally held that a bar was due to the detritus brought down by inland waters, and deposited at a spot where the effluent, by reason

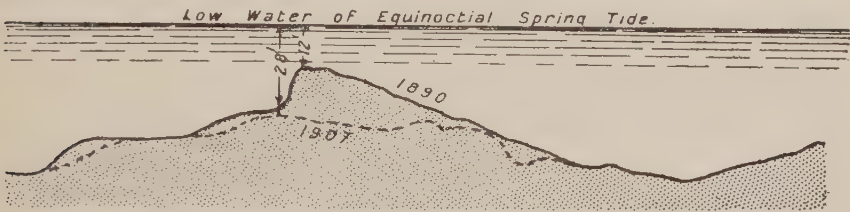


Fig. 251.—Section of the Bar of the River Mersey, showing improvement due to dredging operations.

of its reduced velocity, was no longer able to retain the material in suspension. This argument may indeed hold good in the case of non-tidal rivers, where it has also been advanced to account for the formation of deltas; but in tidal waters the fluctuation of ebb and flow at the river's mouth should obviously result in a dispersal of any such deposit as soon as it had formed, or even before the material had time to settle.

Another view was, that the source of the material being the same, its deposition is brought about by the meeting of conflicting currents, which created zones of slack water. Since, however, the meeting-places of such currents must necessarily vary to a considerable extent from time to time in accordance with the mutable conditions of tide, wind, and weather, the contention does not seem powerful enough on its own hypothesis to account for a fixed bar; and most bars are fairly stationary.

A third theory is that the widening mouth of a river, combined with a constant cross-sectional area, naturally entails a reduction in depth. Against this it is to be urged that bars are almost universally abrupt mounds standing at slopes far steeper than would be the case above water, having regard, that is, to the angle of repose for the material, and that, therefore, they bear no apparent relationship to the much more gradual widening of an estuary.

The opinion now most generally held is that bars are the outcome of littoral drift, and that the chief causes of their formation are tidal currents and storms. Of these, the former agencies are more constant in action, and therefore, perhaps more influential. The flood-tide, travelling along a shore

which is being subjected to secular denudation, carries or rolls along with it a quantity of gravel, sand, and shingle, the motion of which is arrested when it comes in contact with a counter-current issuing from the mouth of a river. This theory does not altogether account for the existence of prominent bars in localities where littoral erosion is not an evident process. In this case, it is contended that the natural tendency of wave motion is to produce irregularities in the bed of the sea, and that these irregularities in certain places have culminated in definite ridges and depressions. But here again the explanation seems to be inadequate, since a bar is a special ridge peculiarly associated with river mouths, and not by any means ubiquitous; though, at the same time, it must be admitted that there are bars in existence off the coast-line, where no river finds its outlet, as, for instance, at Portland Bill.

Finally, it is to be noted that there are bars of indurated material, which are evidently of a permanent character and primeval origin, being due to the denudation of the sea floor and the attrition of its softer portions. Such ridges consist either of rock, tough boulder clay, or conglomerate, and they manifestly constitute features attributable to no transporting agency whatever.

The problem is one attended by difficulties; and it apparently does not admit of a single solution only. In many cases there are indirect causes, some of which are obscure; while in general, the predominant tendencies are recognisable. On the whole, it seems fairly well established that the formation of the majority of bars, especially those in shallow, sandy estuaries, is attributable to the conflict between the external and internal physical agencies, and constitutes the régime under which the forces are in a state of equilibrium, more or less stable. Hence, if bars of this character be removed, there is every likelihood of their recurrence unless special preventive measures be taken. Bars of indurated material, on the other hand, are such as to give no ground for any apprehension of this kind.

Training-works.

For the purpose of training navigable channels, any or all of the following measures may be adopted.

(1) **Training by means of Groynes.**—Groynes are narrow jetties generally of timber, occasionally, only, of stone or concrete, projecting from the bank into the bed of the river at right angles to the direction of its flow. In some cases, the groyne is formed by sheet-piling driven continuously and bound together by horizontal runners; in other cases, detached piles are driven in a straight line so as to form, with longitudinal walings, a series of bays or panels, ranging in extent from 5 feet upwards to 20 feet or more. These bays are filled in with planking, laid horizontally on edge, and spiked to

the piles, or by means of bundles of brushwood bound with wires, the interstices between the bundles being packed with clay and shingle. Bags of sand may be used for the same purpose.

The piles for groynes need not be of any great length; a depth of from 10 to 20 feet into the ground will generally suffice. As regards height, they will advisedly be brought at least to the level of the river bank, and as much above it as will serve to indicate the position of the groyne in times of flood. The body-work of the structure need not be carried higher than ordinary high-water level, if indeed so much as that.

Groynes are spaced at varying distances apart: sometimes at intervals equivalent to their own length, sometimes more or less than that standard, according to the special requirements of each case. They have the effect of warding the current off the bank to which they are connected, and of urging it towards the centre of its bed, and so producing a contraction in width. This contraction or concentration of the flow results in increased scouring action, and consequently in a deepening of the river bed.

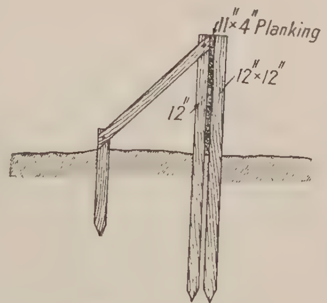


Fig. 252.—Timber Groyne.

As regards the river sides, the spaces between the groynes become gradually warped and accreted, and the accumulation of material intercepted by the groynes leads to the formation of continuous embankments marked by a series of crescent-shaped embayments. These embayments are due to the eddying action of the current, which also has a tendency to denude and undermine the outer ends of the groynes. The extremities, therefore, should be specially protected, or, at least, constructed in a very secure and substantial manner.

Groynes have been extensively used both in this country and abroad: notably on the Clyde, the Tyne, the Tees, and the Danube. They constitute a useful initiatory measure, in that they do not enforce too rigid repression upon a stream. Constructed primarily in short lengths, capable of extension by easy stages, they deflect the current gradually and with an absence of those violent changes of environment which are so liable to produce untoward results.

When accretion has been proceeding for some time, and the current has been induced to occupy its intended channel, the outer ends of the groynes will advisedly be connected by means of a continuous wall so as to form an unbroken front. This leads us on to the second class of work.

(2) **Training by means of Walls.**—The term "wall," though in common use in this connection, is not strictly applicable to the whole class of structures included within its category. In the majority of cases the so-called walls are merely mounds of rubble stone; sometimes the rubble does no more than

form a rough surface paving or pitching to a slope, from 2 to 3 feet thick, or even less; at other times it stands up to some height to a wedge-shaped section with a broad base. Moreover, the wall, whether a pitched slope, revetment, or upright mound, is far from being universally constructed of stone. Fascine mattresses, either singly or in layers, have been most successfully adapted to all the functions of a training wall. Slag and clay are also used, and, in minor cases, bags of sand.

The formation of a **stone training-wall,** though apparently a simple process, is attended by certain difficulties. Rubble, when deposited in a heaped mass, has every disposition to subside in a foundation of soft, saturated sand and mud, particularly when the action is fostered and assisted by the scour of a current along the base. The loss incurred in this way has to be made good, and further material deposited until a firm bearing is obtained; and this result is not achieved, in many cases, without considerable outlay in supplies of stone.

In general practice, the rubble is thrown or tipped overboard by hand

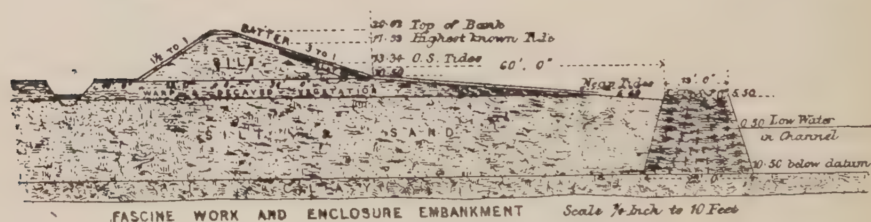


Fig. 253.—Fascine Work in the Wash.[1]

from punts and barges; but the process is slow, and, if the undertaking be at all extensive, it will prove a more expeditious and economical course to discharge from hopper barges. When dealt with in this way, the stone takes an initial slope of $1\frac{1}{2}$ or 2 to 1, which subsequently may become modified to 2 or 3 to 1.

Fascine work has been largely practised as a substitute for stone in cases where the bed of a river consists principally of quicksand incapable of supporting any great intensity of pressure. And as most estuaries are of a sandy nature, more or less uncertain and treacherous, it is a system which naturally suggests itself, in those cases, for adoption. Circumstances are particularly favourable to fascine work, for instance, in the sodden, low-lying land on the shores of the Netherlands, and at the outlets of the fenland on the east coast of this country.

The nature of fascine work has already been alluded to in connection with its employment for jetty and mole construction. For that purpose it is chiefly built up in the form of *mattresses* which are equally suitable for covering a

[1] Extracted from Wheeler on "Fascine Work at the Outfalls of the Fen River," *Min. Proc. Inst. C.E.*, vol. xlvi., Plate 8.

large area of sloping bank, and for being raised in tiers. Where mattresses are not essential, faggots, or "kids," as they are locally called in Lincolnshire, consisting of 6-feet lengths of thorn branches, cut from hedgerows, and made up into bundles 3 feet in girth, may be utilised. These are lighter to lift and easier of manipulation. They are placed overboard, and weighted with sods and clay until they sink, the wall being built up in this way, with the kids overlapping each other in transverse layers.

The interstices of fascines in a waterway rapidly fill with a deposit of earth and detritis, which soon solidifies, and the whole becomes a tough, composite bank, closely cohesive, and, at the same time, fairly flexible; so that if any undermining should happen to take place, no sudden, abrupt fractures would be produced, but the mass would settle uniformly, and no part of it would have any tendency to slip out of position into the fairway of the channel, as sometimes happens in the case of rubble walls. Moreover, the tenacity of brushwood offers effective protection, not only from the ordinary scour of streams, but from the wash of passing vessels and the discharge of heavy rainfalls during periods of low water.

Arrangement of Walls.—Training-walls are either single or double. Single walls only are necessary when the nature of the flow is such that erosion is confined to one side of a river, as is the case at bends. In intermediate positions and straight reaches, and also in places where it is desirable to direct a stream across from one bank to that opposite, two parallel walls are requisite; otherwise the stream will exhibit a tendency to spread, and the channel to shoal.

At the mouths of rivers, double retaining walls may be either parallel or splayed, and the splay may be inwards or outwards, so that the walls either converge or diverge as they approach the sea. Parallel retaining walls serve to maintain the downstream current unimpaired in strength and velocity; but if they are carried up to any height in tidal estuaries, they lead to an accretion which obstructs the flood-stream and excludes a considerable portion of the water which would otherwise enter the estuary. Another danger attaching to such walls is the likelihood of shoaling in the neighbourhood of the entrance, due to the arrest of littoral drift by the walls. This drawback has manifested itself in a number of cases, and at Dunkirk, for instance, the jetties have been extended outwards from time to time, in order to reach deep water and to scour away the intermediate deposit which threatened to destroy the accessibility of the port. Moreover, parallel walls do little or nothing towards the dissipation of storm-waves passing in from the sea. It is from this point of view that converging walls have been designed, the inclosed area being of the nature of a basin containing a relatively larger mass of water, upon which external agitation has less effect. These walls, in fact, are sometimes adapted so as to form compartments called **wave-traps** (fig. 254). The drawback of the system is the same as that mentioned in connection with parallel walls—viz., the reduction in volume, and conse-

quently in scouring efficacy, of the influent waters. This objection, of course, only applies to tidal seas.

From this last standpoint, divergent walls are preferable, for with their splayed arms they admit the flood-tide freely and the outward flow of the ebb maintains the channel in that gradually widening form which is the ideal régime of an estuary. The contraction of the sides must not be too rapid, or there will be a tendency to throttle the inward flow, and pile up the tidal wave until it forms something of the nature of a "bore"—the term applied, in certain rivers, to an influx of water possessing a steep face and moving with considerable rapidity. This is dangerous alike to navigation and to the stability of the banks. It must be admitted that no great uniformity is exhibited in the expansive ratios of natural estuaries. They fluctuate exceedingly, and range, in parts, from something like 2,550 feet to the mile in the Humber to little more than

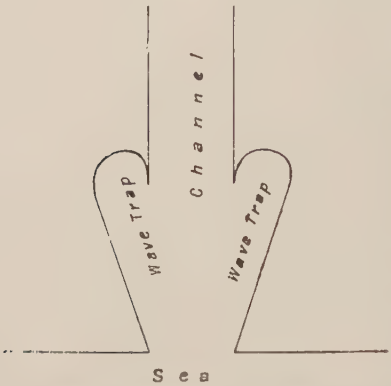


Fig. 254.—Arrangement of Wave Traps.

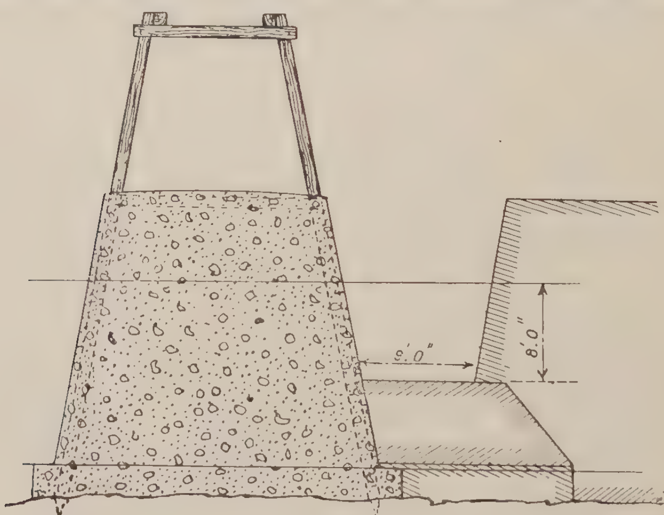


Fig. 255.—Whitby Harbour. Front view of new wave trap.

100 feet to the mile in the Severn. On the whole, however, it may be said that a ratio of 2,000 feet to the mile constitutes a suitable standard for adoption.

Height of Training-walls.—The height to which training-walls should be raised is a moot point. If nothing more than the mere rectification of a

channel be in view, the wall will only be of the nature of a low revetment, confirming and protecting the edge of a newly-formed bank, and need not be raised above low-water level. It has been urged against this, that in a sandy estuary, a channel so formed would soon be silted up with sediment washed in from adjacent banks over the top of the walls. There is, however, no more reason why silting should take place under the new conditions than there is under the old, and it may be safely assumed that the stream is powerful enough to maintain its own bed.

If it be desired to form an entirely fresh channel, or to divert radically an existing one, something more definite than a mere revetment becomes necessary; scarcely anything less than a half-tide wall will suffice to confine a stream within arbitrary limits and guide it through a novel environment. The tendency to resume a long-established course must always remain a powerful influence, if ever the compelling forces be modified or removed.

When land reclamation is definitely aimed at, training-walls will be first laid up to mean tide level, and then gradually raised until the level of highest high water is reached.

(3) **Training by means of Dredging.**[1]—Of all the agencies at work for the regularisation of channels and for the removal of natural impediments, there is none so effective and so powerful as dredging, exemplified, as it is, at the present day by machines of enormous size and tremendous capabilities. Natural scour is serviceable enough in its way, but it is only effective in soft, friable material. It is quite powerless to remove indurated ground within any reasonable time, and it has no influence whatever on huge boulders and rock. To all training-works, of whatever description, dredging is a most useful auxiliary, and there are few ports the entrance channels of which can be maintained without the aid of continuous and systematic dredging. It is, in fact, the recognised medium for the removal of bars and shoals. Nor are its operations confined to any one class of material. Dredging in rock is as feasible as dredging in alluvium, and boulders are removed as easily as sand.

At its first introduction, dredging was carried on by small and insignificant agencies, but the scope of present-day operations has become so vast and extensive as to necessitate the employment of extremely powerful plant and appliances. Such primitive expedients as "the bag and spoon," "the aqua-motrice," and "the rake," except in very insignificant localities, have given way to large and imposing vessels, self-propelled, navigable, and specially equipped with machinery for dealing with something like from one to five thousand tons of material per hour.

The *conditions of dredging* work are exacting. Formidable obstacles are frequently encountered. Not only is the material dealt with of a very uncertain and varied character, ranging from impalpable mud to adamantine

[1] Dredging appliances are fully dealt with and illustrated in Chapter III. of *Dock Engineering*.

rock, and from the most friable sand to the most adhesive clay, but vicissitudes of climate, weather, tide, current, and wind have also to be reckoned with, and operations must generally be conducted in such a way as to cause the least possible disturbance to the existing conditions of navigation. All these matters cause frequent and expensive stoppages and delays. In some cases, the actual useful working time only amounts to one-fourth of the whole year, and it is never safe, under any circumstances, to reckon upon more than 200 working days per annum. A very large proportion of time is taken up with repairs; breakdowns are a common occurrence, and the expense arising from this cause is no inconsiderable sum.[1] Yet, in spite of all these drawbacks, dredging is an institution of untold value. By its means ports are brought into commercial prominence and saved from extinction. No other system can vie with it.

The *principle of dredging*, originally that of digging and dragging, has been extended to include pumping, so that modern dredgers are divisible into two types: first, those in which the action is mechanical erosion, and secondly, those in which it is hydraulic suction. In the most recent machines, both actions are combined.

Mechanical eroders comprise scrapers, cutters, picks, buckets, and grabs, singly or in combination.

Scraping implements, apart from suctional adjuncts, have only a very restricted application. They are intended to disturb and comminute material to such an extent as to render it readily removable by the force of the current. But the power of a current to maintain material in suspension is strictly limited, and it soon becomes laden to its fullest capacity. When this point has been reached, it can absorb no increment without an increase in velocity, and at the first diminution in its speed it deposits a portion of its load. Hence, mechanical scouring rarely produces more than a slight displacement, and it certainly is not capable of sustaining operations on a scale of any magnitude.

Combined with a suction tube and pump, however, it is a most useful agency. Experiments have demonstrated that, with the aid of suitable cutters and scrapers, marl, stiff clay, and adhesive material generally, may be separated and dissected to a degree compatible with its removal by pumping. The cutters employed are, generally speaking, cylindrical in shape, with straight or spiral blades mounted concentrically round the

[1] Twelve months' record of U.S. dredger "Gedney," working at entrance of New York Harbour:—

Actual working time, parts of 112 days, equivalent to	$92\frac{1}{4}$	days.
Work prevented by weather (fog, storm, etc.),	$29\frac{3}{4}$	"
Occupied in general repairs during winter,	154	"
Occupied in minor repairs,	21	"
Lost from other causes,	10	"
Sundays and holidays,	59	
	366	"

extremity of the suction tube. The efficiency of a cutter depends very largely on its design, on the size, number, and shape of the blades and their positions relatively to one another, and to the suction nozzle. Many of the earlier experimental forms were far from successful in their attempts to remove plastic material. The blades became clogged, and a very small proportion of solid matter found its way into the discharge pipe. Substantial improvements have, however, been effected of late years, and a modern suction cutter dredger is quite capable of dealing with the most adhesive and tenacious materials.

Rock-cutting involves dredging appliances of a different type—those allied to the pick or hand-drill. A long, heavy cylinder of steel, fitted with a hard cutting-point, is raised, and allowed to fall by its own weight upon the surface of the rock, which it splinters and pulverises. The hardest rock yields to this treatment, and the blows are repeated until the fragments are reduced to the size of ordinary ballast ready for removal by a bucket or grab.

The **Bucket Dredger** is to be found either in the form of a continuous band of buckets, called the *ladder dredger*, or of a single bucket, worked at the end of a long arm or lever, and called the *dipper dredger*.

The first of these stands foremost in importance. The principle on which it is constructed is that of an endless chain connecting a series of buckets, which revolve continuously around two pivots, or tumblers, at different levels. The buckets excavate material at the lower tumbler, and discharge it into a shoot while passing over the upper tumbler. Dredgers of the ladder type present two varieties: those in which the ladders are centrally situated, and those in which the ladders are set at each side of the dredger.

The bucket dredger can remove sand, clay, shingle, and marl, with equal facility, and it can even deal with the softer kinds of rock. In harder varieties of rock it follows in the wake of blasting operations, or of a rock-cutter. It will lift boulders of a moderate size. A dredger at Bristol, on one occasion, raised a boulder weighing $2\frac{1}{2}$ tons without the least damage to the bucket. Most dredgers working in glacial clay have had some experience of boulder-lifting.

The **Dipper Dredger,** with a single fixed bucket at the end of a long lever arm, is almost exclusively an American type. It is used mainly on river beds and channels where the working depth is not very great; for sea work in deeper and more exposed water, the ladder dredger shows to better advantage. Mounted on a barge, and working either from one end or through a well-hole in the centre, the lever makes a curved upward cut, and the contents of the bucket, after slewing, are dropped into a scow or hopper ranged alongside.

The **Grab** consists of two or more curved plates, or jaws, capable of opening and closing in response to suitable mechanism. It is worked, to a very large extent, with the aid of gravity. Suspended by a chain or chains from the head of a crane jib, the bucket is allowed to fall freely by its own weight, with open jaws, until it buries itself in the ground. The jaws are then

brought together, and the inclosed mass of earth is lifted. The economical scope of grab dredgers is limited to confined situations where other forms of dredger are unworkable.

The pumping principle is represented by one type only—the suction dredger.

The **Suction Dredger** has proved itself to be unquestionably one of the most remarkable contrivances ever devised for the removal of subaqueous material, both in regard to the enormous extent of its output and the low cost of its operations. It is to some extent, of course, a special machine. There are, naturally, conditions and circumstances to which it is not applicable; but they are few. It would be useless to expect it to dredge hard rock or to lift massive boulders. In all other cases, the efficacy of the suction dredger has been demonstrated beyond question.

The suction dredger consists essentially of a continuous pipe or tube, through which, by means of suitable pumping machinery, material is sucked up and discharged, either into a hopper forming part of the vessel itself, or into a scow ranged alongside, or through a shoot or tube leading to an adjacent bank or shore, which last arrangement lends itself very conveniently to land reclamation purposes. In the case of sand and light material, no preliminary treatment is necessary, but clay and marl have to be disintegrated by the cutters already alluded to, before they are in a condition to be drawn up the tube.

In exposed situations, such as prevail along the seacoast, the suction dredger possesses a marked advantage over apparatus of other types, the working of which is often materially interfered with by the motion of the waves. Equipped with telescopic pipes and flexible joints, the suction dredger adjusts itself to the rise and fall of the sea, and is independent of moderate variations in level, either momentary or prolonged.

There is a great deal to be said, in extension of the foregoing remarks, on the relative advantages of the various types of dredgers, and there are many interesting features in connection with their working which might usefully claim our attention; but space will not permit us to pursue the matter further here.

We turn now to the last item in our series.

(4) **Training by means of Sluicing.**—The principle of sluicing is based on that of the ebb-tide current, which, flowing out of a coastal indentation, scours its passage as it goes. The application of sluicing, however, is restricted to channel deepening and maintenance. It is rarely, if ever, employed in channel-making.

In practice, a large basin or receptacle is provided, within which the tidal water, entering up to the time of high water, is impounded and subsequently discharged through sluices or outlets at or about low water. The most effective period for sluicing is during spring-tides, when the flood waters are large and the ebb level is low.

The method has been largely used in ports bordering on the English Channel and the North Sea, such as Dunkirk, Calais, Boulogne, Dieppe, Ostend, etc., where the discharge of a large volume of water in this way has been found highly serviceable in keeping the harbour entrance channels free from silt. The system has its drawbacks. The retaining basins tend to silt up themselves during the quiescent period of retention. To obviate this, the basin is, in some cases, as at Honfleur, only filled about the time of high water, when the influent is comparatively clear. In other cases, as at Ramsgate and Dover, the basin has been divided into two compartments, one of which is used periodically to cleanse the other.

Some harbours are equipped with a natural sluicing basin. Such is the case at Santa Ana, Curaçao, which is probably one of the finest natural harbours in the world. The Schottegat lagoon, behind it, forms a tidal basin $2\frac{1}{2}$ miles in length, with a depth of 50 to 60 feet. At Yarmouth there is a magnificent backwater, receiving various tributaries and forming an immense reservoir of fresh and salt water, which serves to keep the harbour fully open, and even deepens the approaches.

In cases where the sluicing basin is fed with fresh water, it is desirable to note that the specific gravity of fresh water being less than that of salt water, there is a marked tendency for the lighter liquid to flow over the denser; and this phenomenon, which is a matter of ordinary observation, detracts somewhat from the scouring effect of fresh water.

A coastal inlet or estuary may be transformed into an automatic sluicing basin by the construction, as at the mouth of the Liffey, of a low retaining wall, which becomes submerged above half-tide level. When the tide falls again below this level, the ebbing water converges to a contracted outlet, which sluices the harbour entrance.

Compared with dredging, sluicing is an agency not nearly so powerful or so effective. The head or pressure under which it acts is rapidly dissipated by the resistance which it encounters, and at some little distance from the source its scouring effect is greatly reduced, and rendered but slightly appreciable. Indeed, it may be said that sluicing, as a means of channel maintenance, has practically been entirely superseded by dredging.[1]

[1] There are, of course, exceptional cases where the circumstances are favourable to sluicing as an auxiliary agency. Speaking in 1911, Sir John Purser Griffith, then Chief Engineer to the Dublin Port and Docks Board, said: "With regard to Dublin, through the great works constructed in the early part of the nineteenth century, about 10 feet had been gained in depth over the bar. About the year 1880, the bar seemed to come to a standstill, and it fell to his lot to advise as to what could be done. There were various proposals for narrowing the entrance and increasing the scour, and also for increasing the tidal volume thrown on the bar; but with the knowledge already gained he thought himself justified in recommending the Port Board to cut through the bar, so as to have 20 feet at low water, and he believed there was sufficient scouring power behind to maintain the deepened channel. He was glad to say that, as far as the work had gone, his forecast had been fulfilled. The channel had been deepened to 20 feet at low water by dredging, and since dredging was stopped there had been a further increase in depth, due evidently to the scouring power that produced the first improvement."—*Min. Proc. Inst. C.E.*, vol. clxxxiv.

So far as it is possible to deal in a single chapter with a subject which is capable of being expanded to an entire volume, the foregoing represents an attempt at a fair review of the methods by which the entrance channels of ports are improved and maintained. It only remains to append a few examples of work actually carried out in various parts of the world, in order to afford some illustration of the manner in which general principles are applied to particular cases, and the modifications which have to be introduced to meet special local conditions.

Instances of Channel Regulation Works.

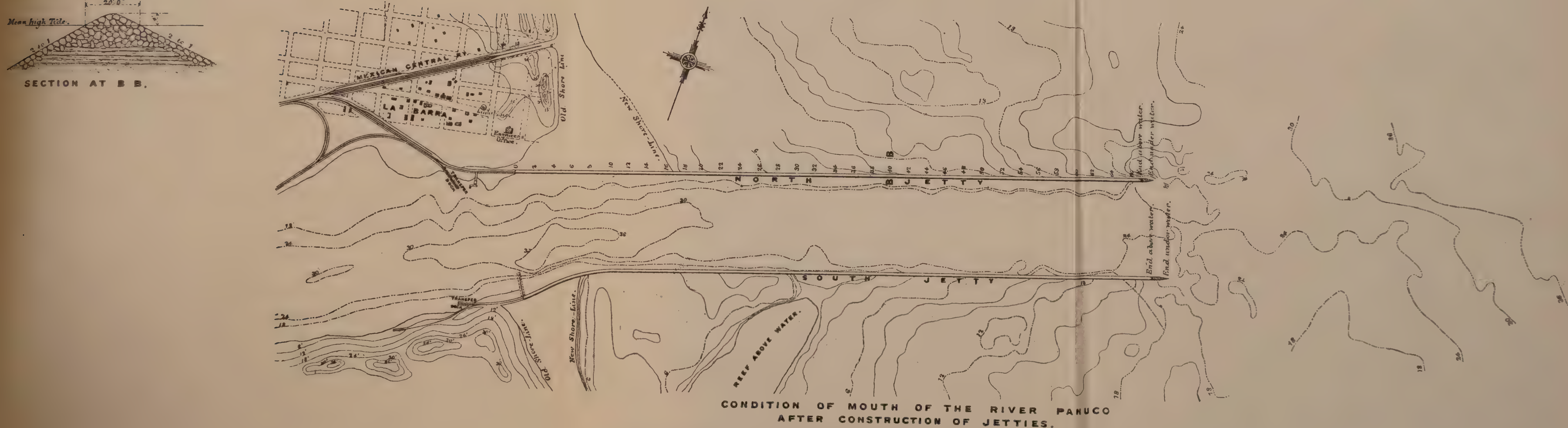
Regulation Works at the Mouth of the River Weser.[1]—The estuary of the Weser has been undergoing a course of improvement since the year 1891, when Herr Franzius designed works consisting chiefly of two training-walls for the removal of a bar, caused by a division of the current, which had existed for about thirty years, and had finally attained a length of nearly 7 miles. One training-wall was situated on the left bank about $4\frac{1}{3}$ miles long, opposite Bremerhaven, and lying between that port and Imsum, and the second wall was on the right bank, 1 mile long, between Imsum and Wremen. These walls had the effect of reducing the excessive width of the river, and the increased velocity of the water produced a scour which removed completely 7 million cubic yards of sand in two years, and changed the position of another $4\frac{3}{4}$ million cubic yards—all in a length of 12 miles. The movement of material, however, simply led to the formation of banks elsewhere, and a suction dredger of 500 I.H.P. was ultimately obtained to keep open the channel to Bremerhaven for deep-draughted vessels, which it has done in a perfectly satisfactory manner. A second dredger of greater capacity was added, in 1898, as a substitute in case of damage to the earlier boat, possibly entailing lengthy repairs, and also in order to be in a position to deal with simultaneous shoaling of distant parts of the estuary.

The Robben Plate divides the outlet of the Weser into two streams, and there is a further subdivision at the West Ever Sand. In consequence of these obstructions, an outer bar had formed at the north end of the right-hand channel. To remedy this defect, in 1896 the branch stream between the East and West Ever Sands, which has a breadth of about two-thirds of a mile, was dammed by depositing a layer of weighted fascines, $3\frac{1}{4}$ feet in height, and in the following year a second layer was added. Simultaneously, protection works were carried out along the frontage of the Wurster Watt. The object in view during these operations was to attain a low-water depth throughout of 26 feet, so that there might be no restriction to the passage of deep-draughted vessels. This object was in due course attained.

It was also at one time proposed to block the channel lying to the left of

[1] Franzius and Thierry on River Regulation Works in Germany, *Min. Proc. Inst. C.E.*, vol. cxxxv.

Fig. 256.—Training Works on River Weser.



Figs. 257 and 258.—Training Jetties at Tampico Harbour.

the Robben Plate by means of a training wall and dam, as indicated in fig. 256. The works, however, have not been carried out, as the channel has manifested a natural tendency to silt up, and it is considered that any artificial assistance in this direction is superfluous.

Writing in 1906 to the author, Herr de Thierry remarked :—

"The works begun in 1896 and 1897 between the East and West Ever Sands, and for the protection of the Wurster Watt, have been carried out and maintained. The two dredgers, "Columbus" and "Franzius," have been nearly constantly at work deepening and regulating the channel. The dredging of a deep channel caused difficulties between Imsum and Wremen, owing to a bank of boulders bedded in very stiff clay. The dredgers have worked chiefly at this place, and lower down, where the velocity of flow is sensibly decreased, on account of the branching off of the stream between the East and West Ever Sands. A low-water depth of 28 feet throughout was, however, notwithstanding these difficulties, achieved two years ago, and has since been easily maintained."

Training-Jetties at Tampico Harbour, Mexico.[1]—The improvement works at the mouth of the River Panuco, for the port of Tampico in Mexico, consist of two parallel jetties which have been built out from the shore-line into the Gulf of Mexico. They are about 6,700 feet long, and extend into 24 feet of water, their direction being E.N.E., and they are 1,000 feet apart between centre lines. Following the precedent of the work at the mouth of the Mississippi, Dr. Corthell, their designer, constructed them of brushwood mattresses consolidated with rubble stone and detritus. The brushwood was obtained locally, either from the adjacent banks of the river, in which case it was conveyed by barges, or from near the railway, when it was transported by waggons provided with side-posts to retain the material, which, of course, though light, was bulky. The railway was specially extended from the town of Tampico to the mouth of the river for the purpose of conveying materials not only to the site, but also to their place of deposit. To this end, a trestle pier was constructed, which carried a double line of rails with several cross-overs. The mattresses were slung from the pier, between the underside of the pile caps and the surface of the water.

"For building the mattresses, supports of pine scantling, about 3 inches by 8 inches and of a length equal to the width of the mattress, were suspended athwart the jetty line from the caps and stringers of the pier, by means of ropes so arranged that they could be easily and simultaneously released. On the skids were laid other lines of scantling 3 inches by 6 inches, for about 60 feet, the length of the mattress lying longitudinally with the jetty. In these scantlings, forming the bottom framework of the jetties, there were inserted, before being laid on the skids, iron rods $\frac{3}{8}$ inch in diameter and of the length required for the thickness for the mattress, which ranged between 4 feet to 7 feet. These longitudinal strips were placed 5 feet apart on the

[1] Corthell on Tampico Harbour Works, *Min. Proc. Inst. C.E.*, vol. cxxv.

suspended skids, with the rods upright; the brush was then brought to the work, either in a barge alongside when the sea was smooth, or by cars overhead if the sea was rough. It was packed as closely as possible, first in a layer athwart the jetty, and then in a layer lengthwise with the jetty, and so on, until the required thickness was obtained. Mattress strips, or scantlings, of the width of the mattress, were then placed over these rods; and by means of heavy mallets and powerful 'grip' levers, with an iron jaw to take hold of the rod, a pull of 3,000 lbs. was brought to bear, and the mattress was compressed about 20 per cent. The rods were then bent down over the strips to hold them securely."

The character of the brush was not altogether satisfactory; it was generally crooked and very stiff, and did not yield to compression during construction, or give way to form solid work until it had been heavily loaded a long time with stone under water. For this reason, the final compression after loading was nearly 50 per cent. of the bulk of the mattress on completion.

"Between six and twelve waggon loads of riprap stone, each car carrying about 12 cubic yards, were then usually hauled by the locomotive to the point over the mattress. The ropes suspending the mattress were released, and the stone from the waggons thrown on to it, causing it to sink out of sight in a few moments. Mattress work was thus carried on when it would have been impossible to do so with a floating equipment. By this method of construction, which, with some change in details, was followed throughout the work, the mattresses were built round the piles, of which there were between four and eight in each bent, the bents being 15 feet apart. The only modification was in the varying thickness and width of the mattresses, and in often substituting ½-inch rods for the smaller rods in the corners or outer sides. Only one or two of the mattresses were injured by the waves."

As the south jetty was on the opposite side of the river from the railway terminus, it was necessary to ferry the waggons of rock and brushwood across. This was done by a "model" barge, with two tracks holding six waggons, aprons being arranged, adjustable to the tide, at the end of a short pier on each side of the river. A locomotive for hauling the waggons on the south side was ferried over, and used between the barges and the work.

The total amount of brushwood used on the jetties until the close of their construction in 1892, was 390,532 cubic yards; of rock, 373,048 cubic yards; and of pine piling, 253,347 lineal feet.

Dr. Corthell furnishes the following details of actual cost:—

	s.	*d.*	
Uncreosoted piles from the United States, .	1	4½	per linear foot.
Uncreosoted Palma or other approved native piles,	0	11	,,
Creosoted piles from the United States, . .	2	5½	,,

	s.	d.	
Mattress work, ...	6	2	per cubic yard.
Brush work, ...	4	7½	,,
Large stone (not exceeding 3 cubic yards), .	9	3	,,
Small stone (not exceeding ½ cubic yard), .	7	5½	,,
Concrete blocks, ...	41	8	,,

The prices include not only the materials named, but also all iron, straps, fastenings, ties, scantling, framework, etc., required for their use.

Wave Basin at Westport Harbour, New Zealand.[1]—"Westport Harbour is situated at the mouth of the Buller River, on the west coast of the middle island of New Zealand, and is the most important coal port of that colony. The river discharges into the Tasman Sea, nearly at right angles to the coast-line (*vide* fig, 259, p. 316). About 6 miles to the westward of the entrance a natural shelter from the prevailing south-westerly winds is formed by Cape Foulwind and the Steeples."

The external works, designed by Sir John Coode and completed in 1893, consists of two converging breakwaters of granite rubble affording an entrance width of 700 feet in the clear. The breakwaters were splayed inwardly in plan, in order to provide wave-basins on each side of the river for the dissipation of heavy seas during gales. It has been found, however, by Mr. Rawson, the engineer to the Westport Harbour Board, that the basin on the west side was unnecessary, and a training-wall, the expense of which might have been saved if the western breakwater had been run straight to a point 1,000 feet higher up the river bank, has since been constructed. "The entrance is open to gales from the north-north-west to the north-north-east, but the heaviest seas are experienced from the north-west; in exceptional cases, occurring perhaps not once in a year, these seas break about one mile out in about 8 fathoms of water, and from thence to the bar is a mass of broken water. On ordinary occasions, however, the break is close to and on the bar; and in what is considered a very heavy sea, the waves range between about 10 feet and 12 feet in height." From whatever quarter they come, the seas are broken on the shoal within the harbour and completely dissipated in the eastern wave basin.

Entrance to Richmond River, N.S.W.[2]—The Richmond River is one of the most important rivers in Australia, serving a large agricultural and farming district. Owing, however, to the cultivation carried on along its bank, the flood waters are heavily charged with silt, and this naturally results in considerable deposits in the vicinity of the entrance. This deleterious action is somewhat counteracted by occasional heavy floods, which scour the channel, although only a small percentage of the rainfall over the basin finds its way into the river, owing to the permeability of the soil and

[1] Rawson on Westport Harbour, *Min. Proc. Inst. C.E.*, vol. cxxxvi.

[2] Burrows on Improvements at Entrance to Richmond River, *Min. Proc. Inst. C.E.*, vol. clx.

the natural reservoirs formed by swamps. The great drawback to the Richmond River has been the shifting character of its entrance, combined

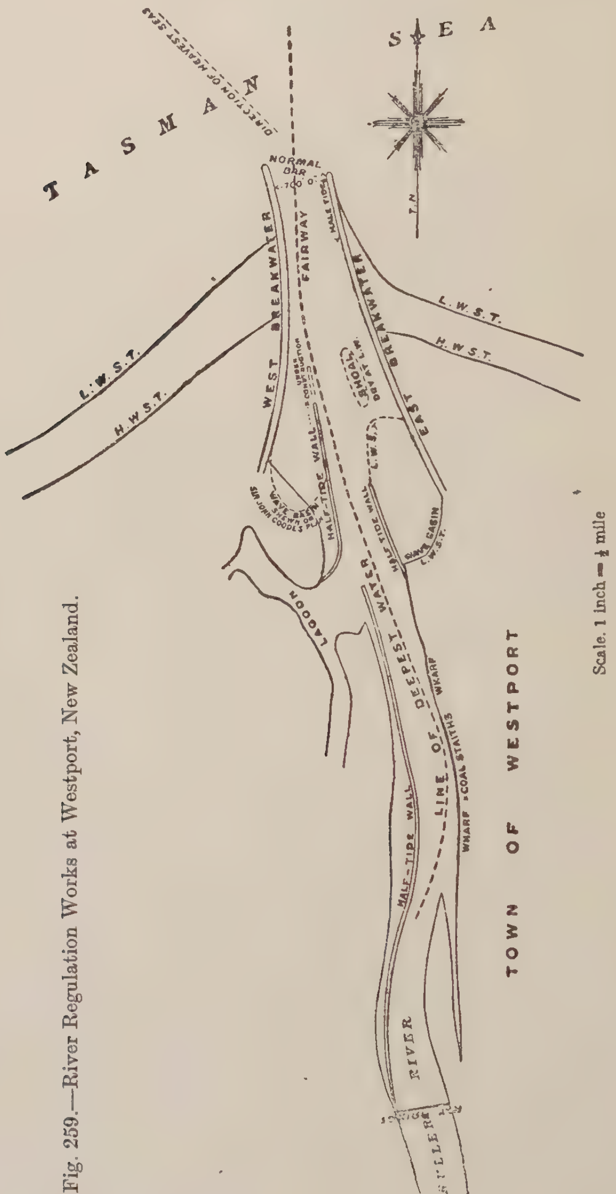


Fig. 259.—River Regulation Works at Westport, New Zealand.

with a shallow bar and adjacent shoals. These evils were intensified by the conflict of the waters of North Creek with those of the main river at their point of meeting, between East and West Ballina.

Fig. 260.—River Training-works at Mouth of Richmond River, N.S.W.

Note.—The width of entrance, as actually executed, is 1,100 feet, instead of 1,000 feet as shown in fig. 260, and the distance between the ends of the breakwaters is 1,170 feet.

The position of the entrance has shifted through a distance of more than $1\frac{1}{4}$ miles, and it has also been noted that floods have caused the channel to break through the Southern Spit on four occasions in thirty-five years. Thus, the navigation of the port has been dangerous at all times, and on many occasions impossible.

In 1888, Sir John Coode was invited to report upon such means as were available for fixing the channel and regulating its width, so that the scour might be confined to a definite track of proper proportions ; to neutralise the obstruction offered by certain rocks near the mouth of North Creek ; and to prevent the conflict of the waters from the North Creek with those from the main river. The remedial works executed are shown in fig. 260. Some of these works have been carried out as designed ; others have been somewhat modified in accordance with experience gained during the course of operations. The main features are sufficiently intelligible, and the only point calling for particular notice is the somewhat unusual addition of a middle training-wall.

"The construction," says Mr. Burrows, "of the new middle training-wall, which reaches ordinary high-tide level for the greater part of its length, was determined upon by Mr. Darley, then engineer-in-chief, at a time when the unfinished condition of other works in progress allowed a large sand-spit to form across the area between the mouth of North Creek and the south wall, and it was found necessary to train the tidal currents of the river at this place, so that the discharge at ebb-tide would tend to prevent the spit increasing the obstruction to navigation at the river entrance. Gaps or openings were left in the wall for the preservation of the old navigable channel along the south wall, until a new channel should be dredged along the north side of the middle wall." The results obtained have proved satisfactory.

The dredging of a channel through indurated sand affords an interesting comparison of the relative efficiency and economy of a suction dredger and a ladder dredger respectively, both working in the same material. The suction dredger "Dictys" was fitted with revolving cutting gear, without which she would have been useless, and the spoil was deposited over an adjacent wall, by means of pipes laid on pontoons, into a blind channel which had to be filled up. The bucket dredger "Alcides" followed in the wake of blasting operations, which were carried out cheaply and expeditiously, as the material was easily bored by a water-jet working at a pressure of about 80 lbs. per square inch, and a hole could be put down in this soft rock at the rate of 1 foot per minute. Nobel's Glasgow dynamite was used, and four holes were exploded simultaneously by an electric battery, the ladder dredger following and lifting the débris into hopper barges for conveyance elsewhere.

The following is a tabular statement of the result of a period of working extending over a year :—

Name of Dredger.	Type of Dredger.	Period of Test.	Effective Work Done.	Rate per Month.	Cost per Month.	Cost per Cubic Yard.	Minimum Rate per Cub. Yard per Month
		Months.	Cub. Yds.	Cub. Yds.	£	s. d.	s. d.
"Alcides,"	Single ladder, with boring-punt, diver, explosives, and tugboat,	12	17,580	1,465	400	5 5½	3 9
"Dictys"	Pump with cutting gear,	15	15,163	1,011	250	4 11¼	3 0

The comparison as regards output lies in favour of the bucket dredger; but as no towing of dredged material was necessary in the case of the suction pump, the cost of working by the latter system was lower. Possibly it might have been lower still, as at the commencement of the period some experimental work was being carried out with various kinds of blades in the cutting gear, to ascertain the most effective form, with the result that the original cutters were retained with but slight modification. The cost given in the statement includes wages, stores, repairs, etc., but excludes any interest on capital cost or charge for depreciation.

Entrance Channel to the Port of Ostend.—The fairway leading to the quays of the port of Ostend is maintained by a dual system of dredging and sluicing. Up to the year 1898 sluicing alone was in vogue, and its effects were deemed satisfactory and adequate. This was perhaps more particularly the case in the interior of the channel, which was subject to silting of a very light nature, the material being chiefly mud. At the entrance and in the external fairway, the results were not quite so pronounced, owing to the more compact and sandy nature of the deposit.

When, in 1898, new works were undertaken for the development of the accommodation of the port, two out of the three existing sluicing basins were withdrawn from use, and have since been demolished, their sites being used for other purposes. A new sluicing basin of much larger area has been designed to take their place.

At the same time, it was recognised that with the increased depth required for modern shipping it would be impossible to realise an effective maintenance service by means of sluicing operations alone. Dredging, therefore, was introduced as an auxiliary. The peculiar conditions appertaining to the port of Ostend are thus set forth by Mr. Van der Schueren in his communication to the International Navigation Congress.[1]

"We have pointed out that the method adopted at Ostend for preserving the navigable depth of its channel, consists of a combination of sluicing and dredging.

"It may be objected that what can be obtained by sluicing can be equally

[1] Van der Schueren on Curage des Ports Maritimes, *Proc. Int. Nav. Cong. Dusseldorf*, 1902.

well obtained from dredging, and that it is not necessary to have recourse to the combined system. At most, it would be a question of cost. It would be necessary to ascertain whether the joint system is more economical than that of dredging alone. Yet, it is not certain that from this particular point the advantage would lie with the combined system.

"But, in our opinion, the preceding considerations are of secondary importance, and ought not to furnish a basis for the solution of the problem.

"In point of fact, under the conditions in which the sluices were installed at Ostend, these latter not only serve to maintain the inner channel, but also, and specially, they maintain the deep berth in front of the new tidal quay, where navigation requires 26 feet of water at low tide.

"Owing to the prevalence of mud in the port, a rapid diminution in depth may be expected to take place, unless very powerful counteracting agencies are brought into play, combating the silting tendency without relaxation or discontinuance.

"In default of sluices, dredging would be essential at the foot of the tidal quay; this would entail the occupation of the quay berth by cumbersome vessels, as inconvenient from the point of view of navigation as from that of trade.

"There is, therefore, every reason for limiting dredging operations at the tidal quay, and, from this point of view, sluicing has its advantages. It reduces the inconveniences to a minimum by considerably diminishing the quantity of material requiring to be dredged."

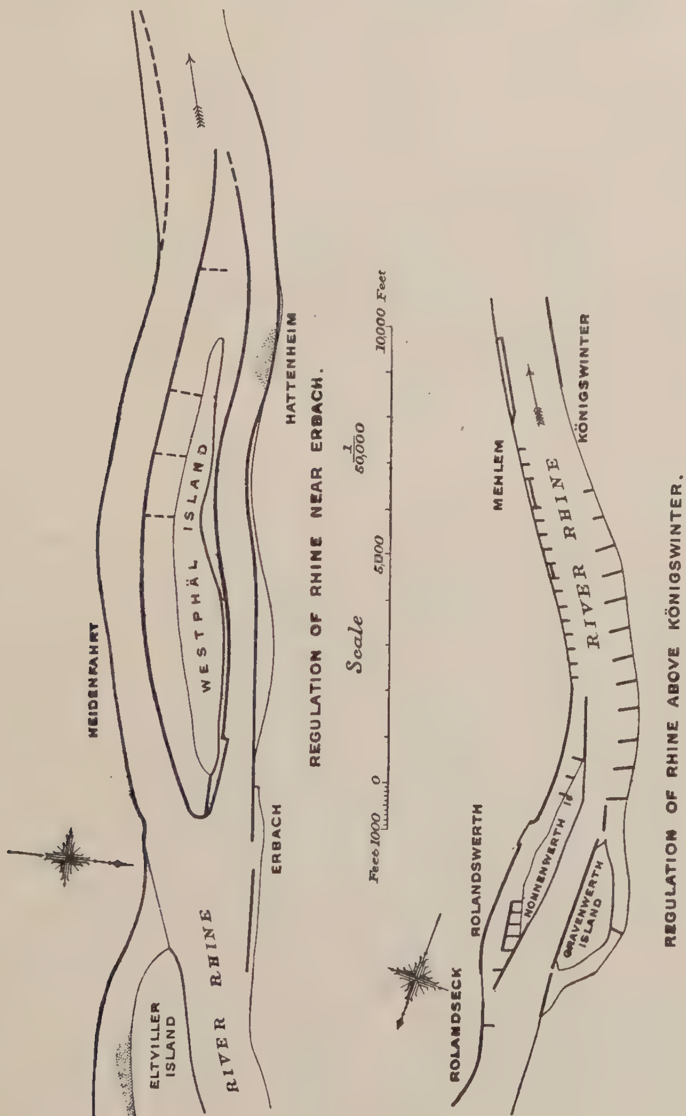
The new sluicing basin has an area of nearly 200 acres, and its contents are discharged through six openings each 16 feet 6 inches in width.

Regulation Works on the Rhine.[1]—"The Rhine, between Mainz and the Dutch frontier, has been systematically regulated in wide, shallow reaches, chiefly by projecting spurs or dykes, generally extending into the river from one bank, but occasionally from both banks where the conditions are unfavourable, as, for example, in a wide reach between two bends; and sometimes longitudinal dykes have been resorted to, connected usually with the river bank by cross dykes. These dykes consist chiefly of earthwork mounds, protected on the face by rubble or pitching, with a rubble mound on the exposed toe; and occasionally fascines are employed in conjunction with stone, or a rubble mound alone. These regulation works have, for the greater part, been gradually carried out during the latter half of the nineteenth century; and two examples of somewhat recent and extensive regulation works, constructed about half-way between Biebrich and Bingen and just above Königswinter respectively, are illustrated in figs. 261 and 262. These works, in conjunction with dredging, where necessary, have provided a navigable depth, at the ordinary low stage of the river, of 10 feet from the Dutch frontier up to Cologne, and 8½ feet between Cologne and Caub at the base of the steep slope below Bingen; above which point

[1] Vernon-Harcourt on Dusseldorf Congress, 1902, *Min. Proc. Inst. C.E.*, vol. clii.

the available depth at average low water is reduced to $6\frac{1}{2}$ feet, which is maintained up to Philippsburg about $22\frac{1}{2}$ miles above Mannheim."

The Port of New York.—There are two navigable entrances (fig. 10) to the port of New York. One, which is frequented by transatlantic liners and



Figs. 261 and 262.—Regulation Works on River Rhine.

ocean-going vessels generally, is flanked by Sandy Hook on the south and by Coney Island on the north, and opens directly on to the Atlantic Ocean; the other is an arm of the sea hemmed in between Long Island and the mainland, known as Long Island Sound as far as its junction with East River at Hell

Examples of Training-walls.

Locality.	Material.	Method of Formation.	Length.	Cost.	Remarks.
Tampico, Mexico.	Fascine mattresses about 60 feet × 15 feet × 4 feet to 7 feet thick.	Slung during construction from overhead staging and deposited. Weighted with stone.	$1\frac{1}{4}$ miles.	Mattress work, 6s. 2d. per cubic yard; brush work, 4s. $7\frac{1}{2}$d. Large stone, 9s. 3d.; small stone, 7s. $5\frac{1}{2}$d.	
River Weser from Bremen to sea.	Fascine mattresses, $66\frac{1}{2}$ feet × $33\frac{1}{3}$ feet × $3\frac{1}{3}$ feet, bound by galvanised wire.	Made and towed 6 miles to destination by tugs. Sunk in position and weighted with stone.	43 miles.	3s. 9d. per cubic yard for materials; 2s. 8d. for labour = 6s. 5d. total. Stone, 6s. 8d. per cubic yard.	Walls left 6 inches above low water to allow for extension of tidal water, and to ensure against damage by waves and ice.
Seine Estuary.	Chalk rubble from adjacent cliffs.	Tipped from barges and levelled to an even face above low-water line.	..	Stone, 1s. $1\frac{1}{2}$d. per cubic yard, *in situ*.	Walls carried up to high-water level, $6\frac{1}{2}$ feet wide at top. Slopes 1 to 1 on land side and $1\frac{1}{2}$ to 8 to 1 on river side, according to current.
Ribble Estuary.	(*a*) Stone from North Wales. (*b*) Local stone. (*c*) Slag.	Deposited by hoppers and trimmed. Discharged from barges by hand. Discharged from barges by hand.	$3\frac{1}{2}$ miles.	4s. 9d. per cubic yard. 1s. $4\frac{1}{2}$d. per cubic yard. 2s. 9d. per cubic yard.	Sectional area of mound, $12\frac{1}{2}$ square yards. Side slopes, 2 to 1. Base width about 30 feet.
Tees Estuary.	Slag from local iron-works.	Discharged by tipping from hopper barges and also by hand from keels and punts.	24 miles.	$10\frac{1}{2}$d. per cubic yard. Ironmasters paid 4d. per ton for removing their slag.	Sectional area of mound, 26 square yards. Side slopes, 1 to 1. Finished height about 5 feet above original surface.

Gate, and forming the principal route for coasting vessels trading to and from the northern states and the Canadian provinces.

These entrances present features of direct and striking contrast, both in regard to their nature and the means adopted for their amelioration. The main entrance is broad and spacious, and, fronting the Atlantic, is exposed to all its storms. Moreover, it is beset with shoals and sandbanks. Sandy Hook itself is but a low-lying bank, more or less submerged, and varying from time to time in form and extent.

Obviously, for such a régime, the process of suction dredging forms the proper system of treatment, and this has been carried on for a number of years past with eminently satisfactory results. There is now a minimum navigable depth of 40 feet at low water along the Ambrose channel, while the older, or Gedney, channel is also available with a depth of 35 feet.

The tidal current is moderate. At the crest of the bar it rarely exceeds $1\frac{1}{2}$ knots, and within the limits of the inner channels its maximum rate is from 2 to $2\frac{1}{4}$ knots.

The Long Island entrance is characterised by a sinuous course, undergoing frequent and abrupt changes of direction. It is comparatively sheltered, but has to wind its way amid the intricacies of an archipelago of islets and rocky reefs, some of the latter rising above the water level, but many of them totally submerged and fraught with danger to navigation. The currents, moreover, are rapid, reaching at certain points a speed of 10 knots, and eddies are numerous. The impetus thus generated, combined with the irregularities of the course, have been, in times past, the cause of numerous disasters to shipping, particularly in the neighbourhood of Hell Gate, where the stream is deflected at right angles past Hallet's Point, to be split up into a multitude of rivulets amid the hidden reefs which abounded at that point. By means of blasting operations, however, on an extensive, not to say gigantic, scale, the worst of these obstructions have been removed, and the channel is now navigable with comparative ease, and, at any rate, with safety.

CHAPTER XI.

CHANNEL DEMARCATION.

Value of Systematic Demarcation—Regulation and Supervision of Channel Marks—Independence of Authorities—Fundamental Characteristics of Signals—Beacons—Buoys—National Systems—Trinity House Regulations—Design of Buoys—Channel Lighting—Luminous Buoys—Wigham Burner—Pintsch System—Lightships—Suspension of Floating Lights—Luminous Beacons and Lighthouses—Incandescent Burners—Light Concentration—Reflectors and Lenses—Catoptric, Dioptric, and Catadioptric Systems—Range of Light—Identification of Stations—Sound Signals—Audible Buoys.

Importance of Channel Demarcation.—One of the most essential features of a modern port is a clear and systematic demarcation of the channels by which it is approached from the open sea. Whether the channels be long or short, winding or comparatively straight, the necessity is equally and incontrovertibly evident, since, in the absence of such guidance, ships are liable to ground on the shoals and submerged banks which fringe the coast-line of nearly every maritime country. Few ports are endowed by nature with an illimitable stretch of open fairway, and, in the majority of cases, restrictions and precautions of no inconsiderable complexity have to be observed. This is more particularly the case with those ports which lie in deep coastal and estuarine indentations, or inland upon the banks of some navigable river within the range of tidal influence. Fluctuations of depth, combined, in many instances, with the eccentricities and vagaries of currents, are a source of continual concern to the mariner, who has most generally to fall back on special local assistance in order to reach his destination. Yet there are circumstances under which such assistance may not be forthcoming; and, apart from this, there is always the desirability of according harbours and ports the fullest possible measure of safe and convenient access. Too much importance, therefore, can hardly be attached to the proper and effective delimitation of navigable channels.

In a maritime country one would naturally expect to find a matter of such vital interest to the community dealt with on broad and systematic lines, and the methods adopted carried to a very high state of perfection. Uniformity of practice and treatment would appear to be the most obvious of desiderata. Yet it must be confessed that, until comparatively recently, the demarcation of approach channels was regarded, to a very great extent, as a matter of almost purely local importance, and it was largely left in the hands of district authorities with little, if any, attempt at national supervision. The inevitable consequence was a diversity of practice, which served to puzzle

and confuse the navigator rather than to assist him. Each port adopted a system of its own, without reference to the broader interests of the country at large, and different rules and regulations were laid down in various quarters, which oftentimes proved as conflicting as they were involved.

This lack of proper and effective centralisation is, of course, no uncommon feature of British administrative methods, being due, in a great measure, to the spontaneous origin and independent growth of the national institutions. The fact is none the less regrettable, in that while the attendant evils do not always manifest themselves so prominently as to attract public notice, and bring about much needed reform, they almost invariably result in extravagance and confusion. Fortunately, in such cases, the natural trend of events is towards the establishment of a hegemony of some kind or another, even though it be imperfect and ill-defined. This tendency, which is manifestly one to be fostered and encouraged, has shown itself in the present instance.

There are still in existence, within the limits of the United Kingdom, three separate bodies endowed with the control of the lighting and buoying of the British coast-line.[1] These are Trinity House, London, *primus inter pares*, the jurisdiction of which extends from Berwick-on-Tweed round to the Solway Firth; The Commissioners of Northern Lighthouses, who administer the coast-line of Scotland and the Isle of Man; and the Commissioners of Irish Lights, formerly the Dublin Ballast Board, who discharge similar duties in respect to Ireland. Apart from these corporations, however, though under their respective suzerainties, there are numerous local authorities exercising control within the limits of their several boundaries. Thus, the demarcation of the approaches to the River Mersey is in the hands of the Mersey Docks and Harbour Board; while the higher reaches of the same river are administered by the Upper Mersey Navigation Commissioners. The Humber Conservancy Board supervises the Humber and its precincts; and King's Lynn Conservancy, the channels of the Wash. Trinity House, London, controls the fairway of the Thames.

Some little time ago there was held a conference which was attended by representatives of Trinity House, the Admiralty, and other interested parties. At this conference a series of regulations were formulated, and recommended for general adoption by all port authorities in this country. These regulations will be referred to in detail later. It is interesting, however, here to note that this step towards the general standardisation of channel marks has met

[1] In September, 1906, a commission was appointed by the Government to report on the respective functions of these bodies, with a view to some method of co-ordination or amalgamation. The Commission did not, however, find any occasion for radical changes in the constitution of the three Boards, and limited their recommendations to matters of detail and account. Trinity House, London, exercises certain jurisdiction over the other two General Lighthouse Authorities and over Local Lighthouse Authorities in England and Wales as regards establishing new sea-marks or altering those in existence.

with approval and success. In fact, a similar congress, but representing far wider interests, and international in character, assembled in Washington in 1899, and drew up certain principles for the regulation of navigable waterways in general, and these principles have become recognised on the continent of Europe and in America as a definite basis for the establishment of a systematic code of channel signals.

Fundamental Characteristics of Channel Signals.—Dealing with the questions, *ab initio*, it will be evident that the essential features of any satisfactory system of demarcation are—

(1) Conspicuousness, by which the marks or signals may be seen from a considerable distance.

(2) Individuality, by which they may be definitely recognised and distinguished, combined with

(3) Simplicity in regard to their signification, and

(4) Invariableness or unalterability of character.

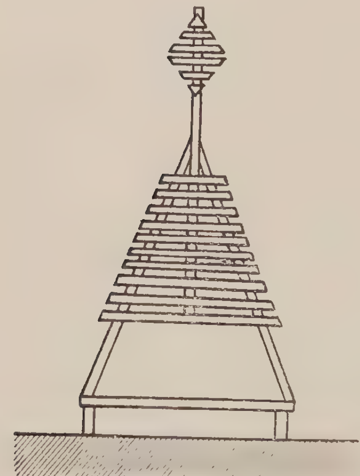


Fig. 263.—Beacon.

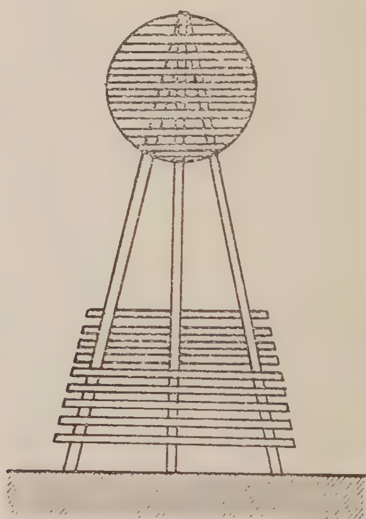


Fig. 264.—Beacon.

In the daytime, these conditions are generally fulfilled by beacons and buoys of definite shape and hue; and at night, by lights of a certain range, intensity, and colour.

To avoid confusion, it will be desirable, as far as possible, to restrict the use of the term "beacon" to fixed, and of the term "buoy" to floating structures. The surmounting signal of a buoy, however, is commonly also designated a beacon. Both buoys and beacons may be used as a means of illumination, but it is solely in regard to their construction and outline that they will be considered in the first instance.

Beacons.—Beacons, then, are prominent objects or structures on the

coast-line or on a river bank, capable of acting as a means of alignment, or as an indication of change of direction. Natural objects, such as lofty isolated trees; topographical features, such as the edge of a cliff, or the summit of a hill; and prominent structures of any kind, such as windmills, factory chimneys, and church steeples, may all be used as beacons. When special erections have to be made, they generally take the form of a wooden framework tapering from a wide base to a narrow top, or forming some distinctive geometrical figure, such as a triangle or lozenge. It is essential, of course, that the beacon should stand out clearly against its background, and the steps necessary to secure this end will vary according to circumstances. One method is to paint the front surface chequerwise in different colours; another to paint it all one colour, and so on.

Buoys.—The difficulties attending the design of a beacon, evident as they are in many instances, are not so great as those involved in the case of a buoy. Steadiness and erectitude are qualities not easily conferred upon floating structures, while the same precision in regard to locality is impossible of attainment. Buoys have to be moored to sinkers, and the length of cable varies from two to three times the maximum depth, which in itself, in tidal situations, is susceptible of considerable fluctuation, so that a buoy is capable of mobility within a circle of not inconsiderable diameter. This renders buoys unsuitable for imparting accurate guidance in regard to alignment. As a general rule, their utility is limited to indicating the proximity of shallows in their immediate neighbourhood.

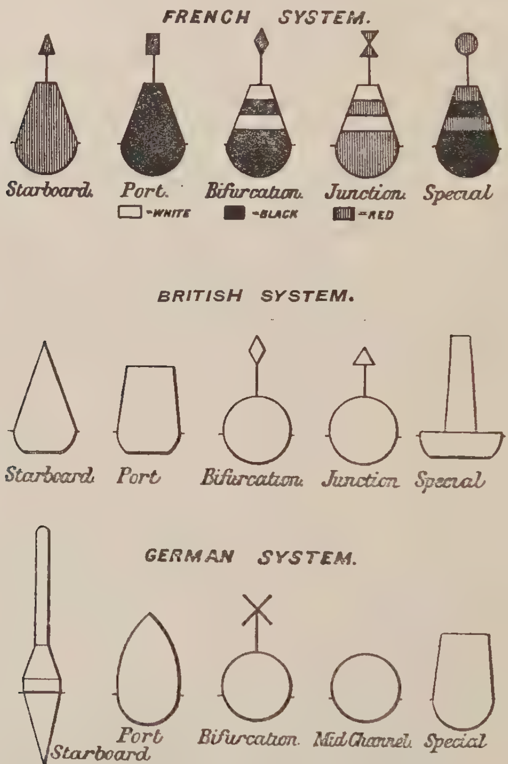
The limiting width of channels is indicated, in fact, by two lines of buoys, one along each boundary. These are termed starboard and port hand buoys, according as they lie to the right or left of the mariner who is approaching a port from seaward. Generally speaking, the maximum distance between two consecutive buoys, on either hand, is a mile or a mile and a half in wide estuaries, and the minimum, perhaps 300 yards in narrow channels, exclusive, that is, of turning-points.

National Systems of Buoyage.—It is interesting in this connection to compare the practice of this country with that of France and Germany. In English practice special stress is laid upon the shape of the buoy structure; more so than in French practice, where colour is every whit as essential as form. Climatic conditions have something to do with this, for on the English coast the state of the atmosphere is often unfavourable to the ready perception of colour at a distance. It is true that colours may be, and are, used in this country as an additional indication, but their use is entirely subsidiary and may vary locally; while in France colour takes precedence of shape. Shape is not entirely disregarded, but the distinction is confined to a surmounting signal, and does not affect the buoy structure as in England. German signals differ from both French and English signals. The series of diagrams in figs. 265 to 267 have been arranged in juxtaposition so as to illustrate the divergencies in type of all three nationalities.

Trinity House Regulations.—The following is a transcript of the regulations adopted in this country in accordance with the uniform system of buoyage approved by the General Lighthouse Authorities of the United Kingdom :—

"1. The mariner, when approaching the coast, must determine his position on the chart, and must note the direction of the main stream of flood-tide.

"2. The term *Starboard Hand* shall denote that side which would be



Figs. 265 to 267.—National Systems of Buoyage.

on the right hand of the mariner, either going with the main stream of flood or entering a harbour, river, or estuary from seaward; the term *Port Hand* shall denote the left hand of the mariner, under the same circumstances.

"3. Buoys showing the pointed top of a cone above water shall be called **Conical,** and shall always be Starboard Hand buoys, as above defined.

"4. Buoys showing a flat top above water shall be called **Can,** and shall always be Port Hand buoys, as above defined.

"5. Buoys showing a domed top above water shall be called **Spherical,** and shall mark the ends of middle grounds.

"6. Buoys having a tall central structure on a broad base shall be called **Pillar** buoys, and, like other special buoys, such as Bell buoys, Gas buoys, Automatic sounding buoys, etc., etc., shall be placed to mark special positions, either on the coast or in the approaches to harbours, etc.

"7. Buoys showing only a mast above water shall be called **Spar** buoys.

"8. Starboard Hand buoys shall always be painted in one colour only.

"9. Port Hand buoys shall be painted of another characteristic colour, either single or parti-colour.

"10. Spherical buoys at the ends of middle grounds shall always be distinguished by horizontal stripes of white colour.

"11. **Surmounting beacons,** such as Staff and Globe, etc., shall always be painted of one dark colour.

"12. Staff and Globe shall only be used on Starboard Hand buoys; Staff and Cage on Port Hand; Diamonds at the outer ends of middle grounds and Triangles at the inner ends.

"13. Buoys on the same side of a channel, estuary, or tideway, may be distinguished from each other by names, numbers, or letters, and, where necessary, by a staff surmounted with the appropriate beacon.

"14. Buoys intended for **Moorings,** etc., may be of shape or colour according to the discretion of the authority within whose jurisdiction they are laid; but for marking submarine telegraph cables, the colour shall be green, with the word 'Telegraph' painted thereon in white letters.

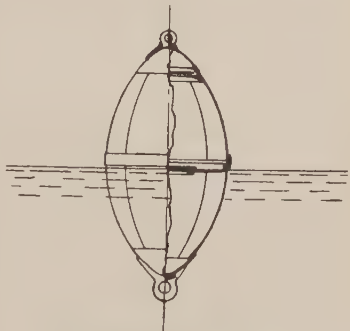


Fig. 268.—Wreck Buoy.

"15. **Wreck** buoys in the open sea, or in the approaches to a harbour or estuary, shall be coloured green, with the word 'Wreck' painted in white letters on them.

"16. When possible, the buoy shall be laid near to the side of the wreck next to mid-channel.

"17. When a wreck-marking vessel is used, it shall, if possible, have its top sides coloured green, with the word 'Wreck' in white letters thereon, and shall exhibit :—

"*By day :* Three balls on a yard 20 feet above the sea, two placed vertically at one end and one at the other, the single ball being on the side nearest to the wreck.

"*By night :* Three white fixed lights similarly arranged, but not the ordinary riding light.

"In foggy weather, when practicable, a bell and gong will be sounded in quick succession, at intervals of not more than a minute.

"18. In narrow waters or in rivers, harbours, etc., under the jurisdiction of local authorities, the same rules may be adopted, or, at discretion, varied as follows:—

"When a wreck-marking vessel is used, she shall carry a crossyard on a mast with two balls by day placed horizontally, not less than 6 nor more than 12 feet apart, and two lights by night similarly placed. When a barge or open boat only is used, a flag or ball may be shown in the daytime.

"19. The position in which the marking vessel is placed with reference to the wreck shall be at the discretion of the local authority having jurisdiction."

Design of Buoys.—The design of a buoy should obviously be such that it will always float upright and be subject to the least possible disturbance of equilibrium in boisterous weather and from drifting ice. Long, narrow buoys, constructed on the principle of the angler's float, are well adapted to withstand currents and rough seas, provided they be not moored from the nether apex, in which case, unless heavily weighted, they tend to heel over considerably. The mooring is preferably attached by a saddle or bridle arrangement about midway on the submerged portion. Elongated buoys are specially characteristic of German practice, on account of the great quantities of floating ice which obstruct the Baltic Sea and its influents during the winter season. Broad-based buoys are suitable for smooth, shallow waters: they take the ground satisfactorily in the event of exceptionally receding tides. In sheltered positions, flat bottoms with rounded bilges make a good arrangement; in a heavy seaway the rounded bottom is to be preferred, or, better still, the hollow cone, as in Admiral Herbert's design. This design gives great stability and reduces the tendency to heel over with the tide; it rides well in rough and choppy waters and maintains its verticality.

Size of Buoys.—Buoys for general purposes may be divided into two classes, according to size. The following table shows the generally accepted dimensions of such buoys:—

BUOY DIMENSIONS.

Type.	First Class.		Second Class.	
	Diameter.	Height.	Diameter.	Height.
	Feet.	Feet.	Feet.	Feet.
Conical, ...	8	10	6	7½
Can, ...	8	8	6	8
Spherical, ...	8	7	6	5¼
Pillar, ...	10	15	8	12

The material used is steel plating about ¼ inch thick. There should be two water-tight compartments in each buoy, so that in the event of collision with a passing vessel, the risk of foundering may be diminished. Mooring-

chains for first-class buoys are about $1\frac{1}{4}$ inch diameter, and they are attached to sinkers weighing about 25 cwts. For second-class buoys, the chains are usually 1 inch diameter, and the sinkers weigh 15 cwts.

The standard adopted for Trinity House buoys is as follows :—

TRINITY HOUSE BUOYS.

	Conical Buoys.		Can Buoys.		Spherical Buoys.	
	Height.	Width.	Height.	Width.	Height.	Width.
	Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft Ins.	Ft. Ins.	Ft. Ins.
First class, .	16 0	12 0	12 6	12 0	12 6	12 0
Second class,	13 6	10 0	10 6	10 0	10 6	10 0
Third class, .	10 6	7 6	8 6	7 0	8 6	8 0
Fourth class,	8 3	6 0	6 6	6 0	6 6	5 9
Fifth class, .	6 9	5 0	5 3	4 9	5 3	4 9

First-class buoys are rarely used, and only in special situations. Those most in vogue are of the second and third classes.

Mooring Buoys constitute a special class of buoys with functions quite outside the sphere of the present chapter. Such reference to them as is necessary is to be found in Chapter IX.

Channel Lighting.—Having dealt with those features of channel demarcation which are available for use in the daytime, we now turn our attention to means adopted for guidance when such signals are no longer naturally visible. Recourse has then to be had to some artificial source of light.

Of the value of luminous signals to the mariner there can be no question. He approaches his destination without reference to day or night, and during the hours of darkness, while in close proximity to land, he is often without any other reliable indication of his position, or trustworthy guidance in his course. At the same time, it must be admitted that in many respects artificial lights, as they exist at present, are far from constituting an ideal system of localisation. The range of visibility is extremely variable under different atmospheric conditions, and in times of dense fog, and even in squally weather, may become of no appreciable value whatever. Then, again, a very powerful light, while serving admirably as a beacon to shipping at a great distance, is a source of some embarrassment at close quarters, dazzling the sight, projecting deep and dark shadows, and obscuring the position of objects which lie outside the illuminated zone, and especially those immediately beneath the source of light. Thus, lighthouses which act as *landfall* or advance lights,[1] form a different class from those which are used to indicate

[1] Lighthouses are commonly divided into four or five classes, according to the particular functions they are called upon to discharge. *Landfall* or *Making Lights* serve as distance signals to indicate to the mariner that he is approaching land, as, for instance, on the Atlantic Route, Fastnet Rock Light, off the South Coast of Ireland ; Cape Race,

navigable channels. In the former case, striking brilliance and extensive range are matters of fundamental importance. In leading and harbour lights,

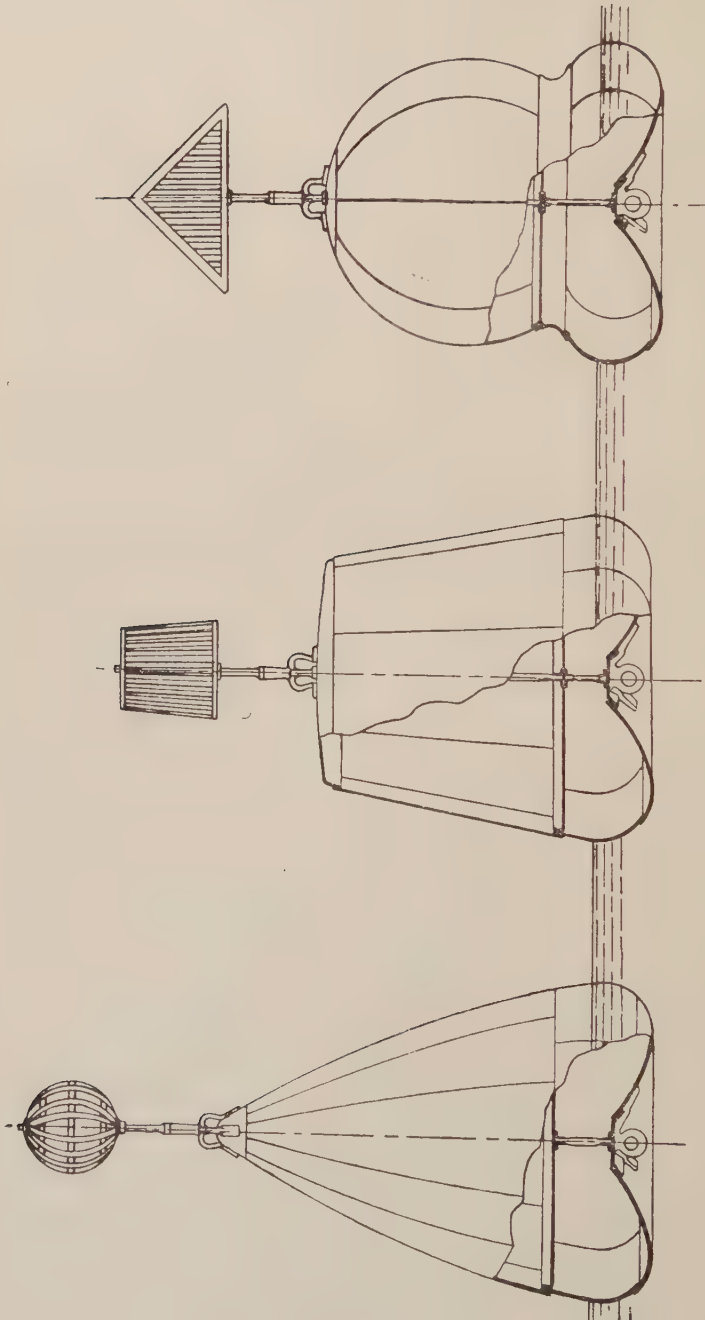


Fig. 269. Fig. 270. Fig. 271.

Trinity House Buoys.


CHART OF LIVERPOOL BAY.
LIVERPOOL
MERSEY
MAD WHARF
FORMBY POINT
CROSBY POINT
LIVERPOOL & SOUTHPORT RAILWAY
FORMBY CHANNEL
FORMBY POOL
FORMBY SPIT
TAYLORS BANK
CROSBY CHANNEL
ASKEW SPIT
GREAT BURBO BANK
GREAT BURBO FLATS
ZEBRA FLATS
QUEEN'S CHANNEL
DREDGED CUT
NORTH BANK
ROCK CHANNEL
MOCKBEGGAR WHARF
SPENCERS SPIT
NEWCOMBE KNOLL
EAST HOYLE BANK
HILBRE SWASH
HILBRE ISLAND
WEST HOYLE
EASTERN BRANCH OF THE DEE
LIME WHARF
WALLASEY
NEW BRIGHTON
EGREMONT
SEACOMBE
BIRKENHEAD
BIDSTON HILL
North Wall Lighthouse
Bidston Lighthouse
Light Fixed White
Leasowe Lighthouse
Light Fixed White
Hoylake Lighthouse
Light Fixed White
Leasowe Castle
Dove Beacon
Grange Beacon
CROSBY LIGHTSHIP
FORMBY LIGHTSHIP
BAR LIGHTSHIP
Light Group Triple Flashing W. every 30 seconds.
KNOLL
B & W horiz.l stripes
Staff & Diamond
DEPOSIT
Horse Channel
Fairway Bell Beacon
Staff & St George's Cross
BIDSTON LIGHT masked to the Eastward of this line S.S.E.½E. (S.28°E.)
LEASOWE LIGHT masked to the Eastward of this line S.S.E.¾E. (S.24°E.)
TRUE NORTH
MAGNETIC
Welshman's Gut
R & W Horizontal Stripes
East Salisbury
R & W Vertical Stripes

Fig. 272.

a much lower degree of illuminating power is all that is necessary or desirable. Few channels present any lengthy stretch of straight fairway; most are characterised by sharp bends and intricate windings, necessitating the employment of numerous lights, which need not have more than a very moderate range, say 5 or 6 miles at the outside.

Channel lighting is effected by buoys, beacons, lightships, and lighthouses.

Luminous Buoys.—Buoys are lighted by means of oil or gas. The former is usually petroleum; the latter either ordinary coal gas, oil gas (including vaporised paraffin and Blau gas) or acetylene. On account of the high pressure required for compact storage, coal gas is unsuitable for consumption as a naked flame, the luminosity of the gas being rapidly reduced with increased compression. The heating power, however, remains almost intact, and, with a mantle, satisfactory incandescence is obtained, though the brilliance of the light and the consequent luminous intensity of the lens are lower than those produced by other agencies. Oil gas under compression does not suffer diminution in illuminating power to the same extent: at a pressure of 10 atmospheres (150 lbs. per square inch) the loss amounts to about 20 per cent. Consequently it is still available for use as a naked flame, while it is also better adapted than coal gas to produce incandescence. Blau gas, which is obtained from the residual products from the process of ordinary oil gas manufacture, produces a much hotter flame than either, and lends itself most readily to the use of the incandescent mantle, to the brilliancy of which it adds in comparison with oil gas some 30 per cent. Speaking broadly, the calorific values of coal gas, ordinary oil gas, and Blau gas are very nearly as 1 : 2 : 3; more precisely, the calories per cubic metre of compressed gas are 4,600, 9,500, and 14,000 respectively. The specific weights compared with air are ·41, ·75, and 1·02. The following is a brief summary of the constituent elements in each, given as percentages:—[1]

	Coal Gas.	Oil Gas.	Blau Gas.
Heavy carbides of hydrogen, . .	5	25	52
Light carbides of hydrogen, . . .	34	55	44
Hydrogen, ...	49	20	2½
Oxides of carbon, ...	12	0	1½
	100	100	100

Newfoundland; and Navesink at the entrance to New York Harbour. *Warning Lights* indicate some particularly dangerous point, or reef, to be avoided, such as Skerryvore, off the West Coast of Scotland, and Eddystone. *Coasting Lights*, for the guidance of coasting vessels, are distributed in sequence along the frequented sections of a coastline to enable a vessel before losing one light to come within range of the next. *Harbour Lights* or *Port Lights* are small lights marking the ends of piers, or jetties, at harbour entrances. *Leading Lights* are those tracing the course of a channel: they are more commonly mounted on lightships or buoys.

[1] De Joly on Lighted Buoys, *Twelfth Int. Nav. Cong. Philadelphia*, 1912.

Acetylene, another gaseous compound of hydrogen and carbon (C_2H_2), with a density of ·92, and more homogeneous than any other, is obtained by the interaction of calcium carbide and water. In the compressed form, it is utilised by absorption in acetone, a liquid hydrocarbon possessing the capability of dissolving, at atmospheric pressure and a temperature of 15° C., 25 times its own volume of acetylene. Compression, except in this form, known as *Dissolved Acetylene*, is illegal owing to the risk of explosion, but dissolved acetylene may be safely compressed to ten atmospheres (150 lbs. per square inch). Under this pressure the gas has a calorific value of 14,000 calories per cubic metre.

The acetone in itself is not a sufficient safeguard against possibility of explosion. The liquid must be absorbed by a porous mass, which will dissipate heat and prevent the transmission of any local disintegration. This porous substance should completely fill the container in order to leave no cavities where free acetylene may collect. The same effect can be produced by mixing the acetylene with an exotermic gas—that is, a gas which absorbs heat when disintegrated. Oil gas is an exotermic gas, and, as such, is capable of admixture with as much as 40 per cent. of acetylene without producing an explosive compound. The addition to oil gas of 20 to 25 per cent. of acetylene has, in fact, been practised on grounds of improved illumination, and for non-incandescent flames and group burners the result was perfectly serviceable, but with the advent of the mantle the occasion for the mixture disappeared.

In addition to the call for a lamp [1] with an invariable rate of supply, the difficulty in using petroleum oil as a direct illuminant lies in the fact that, in the course of combustion, wicks become rapidly charred or coated with carbon, to such an extent as to obstruct and ultimately arrest the capillary attraction necessary for raising the oil from the reservoir to the burning point. This means that the light will be extinguished unless there be someone at hand to dress the wick. Now constant, to say nothing of skilled attention, is impracticable in the case of buoys. They have necessarily to be left to take care of themselves between certain times of inspection, which can only be frequent at the expense of economy; and whether the period be long or short, there is the same risk of failure of the light.

In France, carbonised wicks—*i.e.*, wicks specially prepared by a uniform deposit of carbon upon them—have been introduced. But these, while satisfactory in achieving their special purpose, entail corresponding difficulties of another kind. The wicks not only require careful preparation, but they call for adjustment of the utmost nicety, and they are used with burners of a very complex character. Constant watching is, therefore, still an essential

[1] The term lamp is sometimes applied to the entire illuminating apparatus, and sometimes restricted to a meaning in contradistinction to the burner. In this latter case it refers to the oil receptacle only, and a constant level lamp is one which automatically regulates the supply of oil to the burner.

feature of the system, and this fact discounts their use in connection with buoys.

An English burner, however, known as the **Wigham burner,** has been contrived to meet the conditions of the case, and the following is a brief description of its mode of action.

In an ordinary petroleum lamp, the wick is set perpendicularly to the oil reservoir from which it draws its supplies, and there would be considerable difficulty in making such a wick automatically raise itself as combustion proceeded. Mr. Wigham, therefore, designed his wick to burn horizontally, passing it slowly over a small roller, the light being obtained from the flat side instead of from the end or edge. One end of the wick passes up through an oil-tight brass tube, receiving its supply of oil from the main reservoir [1] by means of feed-holes, and the other end of the wick is brought down through another tube soldered or otherwise secured at its lower end, and standing above the level of the oil in the lamp. A circular float, to which this end of the wick is attached, rests upon the surface of the oil in a copper cylinder at the foot of the lamp. The oil in the cylinder is slowly withdrawn, drop by drop, through a valve of special construction, and the float, in descending with the falling level of the oil, draws the wick in its train, and so causes a constant change in the part of the latter exposed to the action of the flame. The light may thus be arranged to burn without attention for periods of one, two, or even three months. The consumption of oil for both illumination and automatic working, together, is at the rate of about half a gallon per day of twenty-four hours.

Reverting to the alternate system of vapour lighting, a few notes on the method of production will be useful. Oil gas is manufacturable from shale oil, petroleum, or other oils. Heavy oils generally produce a smaller quantity of gas, but of richer quality than light oils. One gallon of oil yields from 70 to 90 cubic feet of gas, and the cost of production per 1,000 cubic feet varies (subject to fluctuations in the price of materials) from 7s. 6d. on a large scale to 15s. on a small one.

As manufactured on the **Pintsch system,** the gas is produced in two ◠-shaped cast-iron retorts, arranged one above the other, connected by a double mouthpiece and set in a suitable furnace. The furnaces are heated by coal, coke, or other fuel, until the retorts have become cherry red. The oil, previously stored in a wrought-iron tank, is pumped into a small vessel, or cistern, near the furnaces, from which it flows by gravitation in a thin stream regulated by a micrometer cock, through a syphon into the upper retort. In order to protect this retort and somewhat retain the oil, a sheet-iron tray is inserted, into which the oil drops and is immediately converted into a dark yellow vapour. Passing through the connecting mouthpiece and along the

[1] Divisions are made in the reservoir to prevent the oil from flooding the wick during momentary disturbance, such as is inevitable in the case of buoys and other floating vessels.

heated sides of the lower retort, this vapour is further decomposed and made into permanent gas, with its impurities. The only outlet of the lower retort is a short pipe, called the descension pipe, through which the gas passes into the hydraulic main, depositing here a certain amount of tar, and thence into a circular condenser. Issuing from a small pipe into the large space of the condenser, the gas cools down and frees itself from the lighter tarry

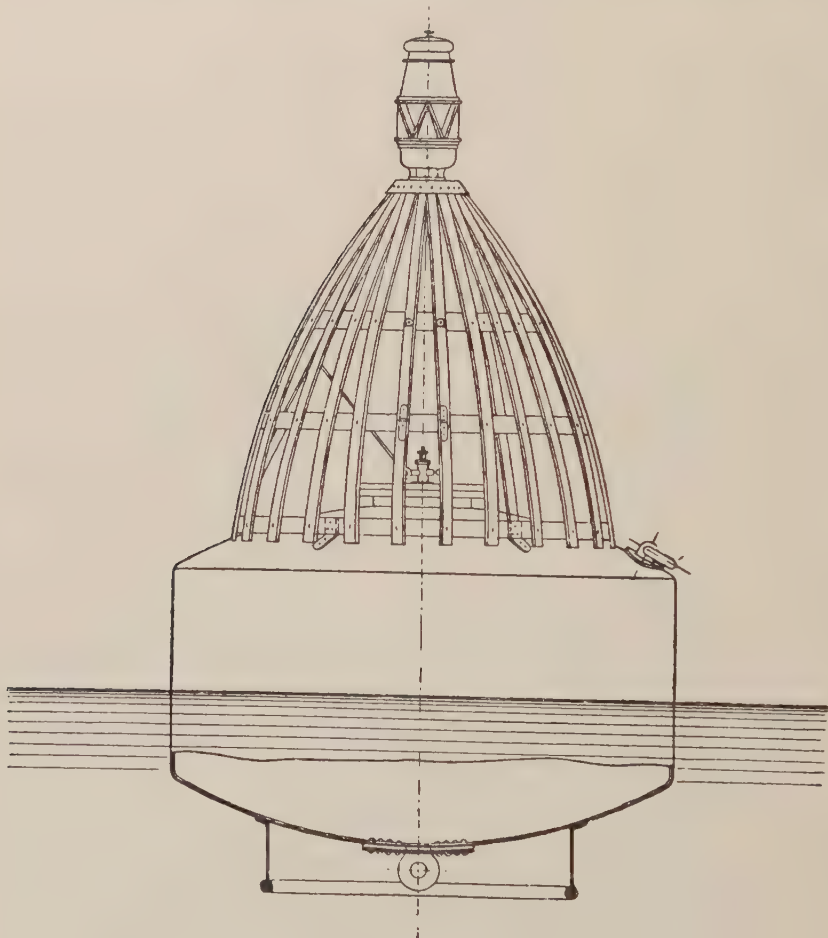


Fig. 273.—Gas Buoy.

matters. It then passes into the washer, where it is forced through about an inch of water, and, afterwards, through two or three layers of lime and saw-dust in the purifier. In small installations, the washer and purifier are generally combined in the same apparatus, one above the other. In the washer and purifier the gas is freed from carbonic acid and sulphur, impurities which are not only detrimental to a good gas, affecting the illuminating

power, but which, moreover, form an injurious deposit in the fittings, regulators, and burners. The gas, when pure, passes through a meter to be registered, and thence to the gas-holder.

The compressors are double-acting pumps worked by steam or by an electric motor, or again, in small installations, by hand. Before the gas is passed into the welded, high-pressure storeholder, it passes through a small vessel and there deposits some hydrocarbon, which is drawn off by a small valve.

The buoy itself constitutes the receptacle for the gas, or a store-holder may be placed within the buoy to be lighted, where it is connected with the burner, and supplies it with gas for periods ranging from two to six months, without further attention.

In the case of gas buoys a special form of construction is rendered desirable. Rivetted joints, however well caulked, tend to leak, especially if the gas be compressed at pressures over 100 lbs. per square inch. Mild steel structures, welded throughout, afford the most satisfactory method of inclosure, and they are, at the same time, less liable to admit water in case of concussion.

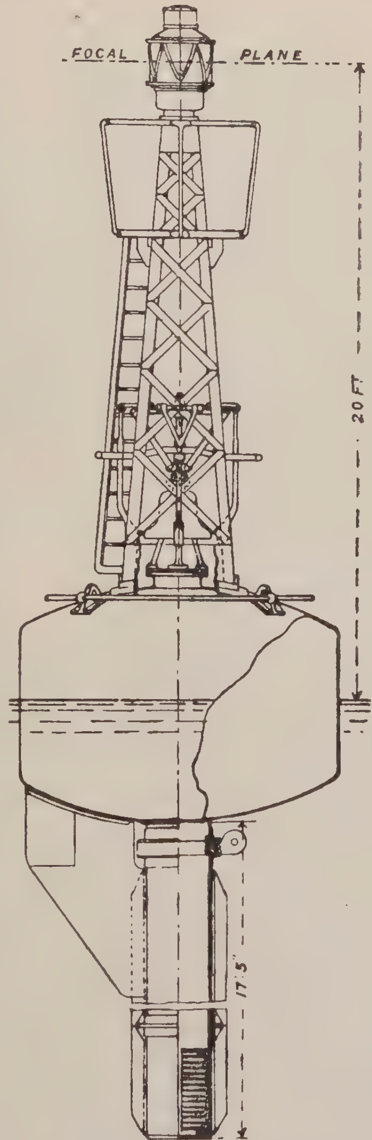


Fig. 274.—High Focal Plane Gas Buoy with Bell used by the Corporation of Trinity House.

Blau gas, so called after the name of its inventor, is obtained by the decomposition at a lower temperature (550° to 600° C. instead of 750° to 800° C.) of the raw or residuary products arising out of the manufacture of ordinary oil gas. After a process of cooling and cleansing by means of water spraying, the gas is liquefied by gradual compression to an intensity of about 100 atmospheres. In the course of compression, which is carried out in two, and sometimes three, stages, the gas parts with those constituents which are easily condensed

and, ultimately, with nearly the whole of its free oxygen. On liquefaction the volume is considerably reduced, and the gas is then put into steel

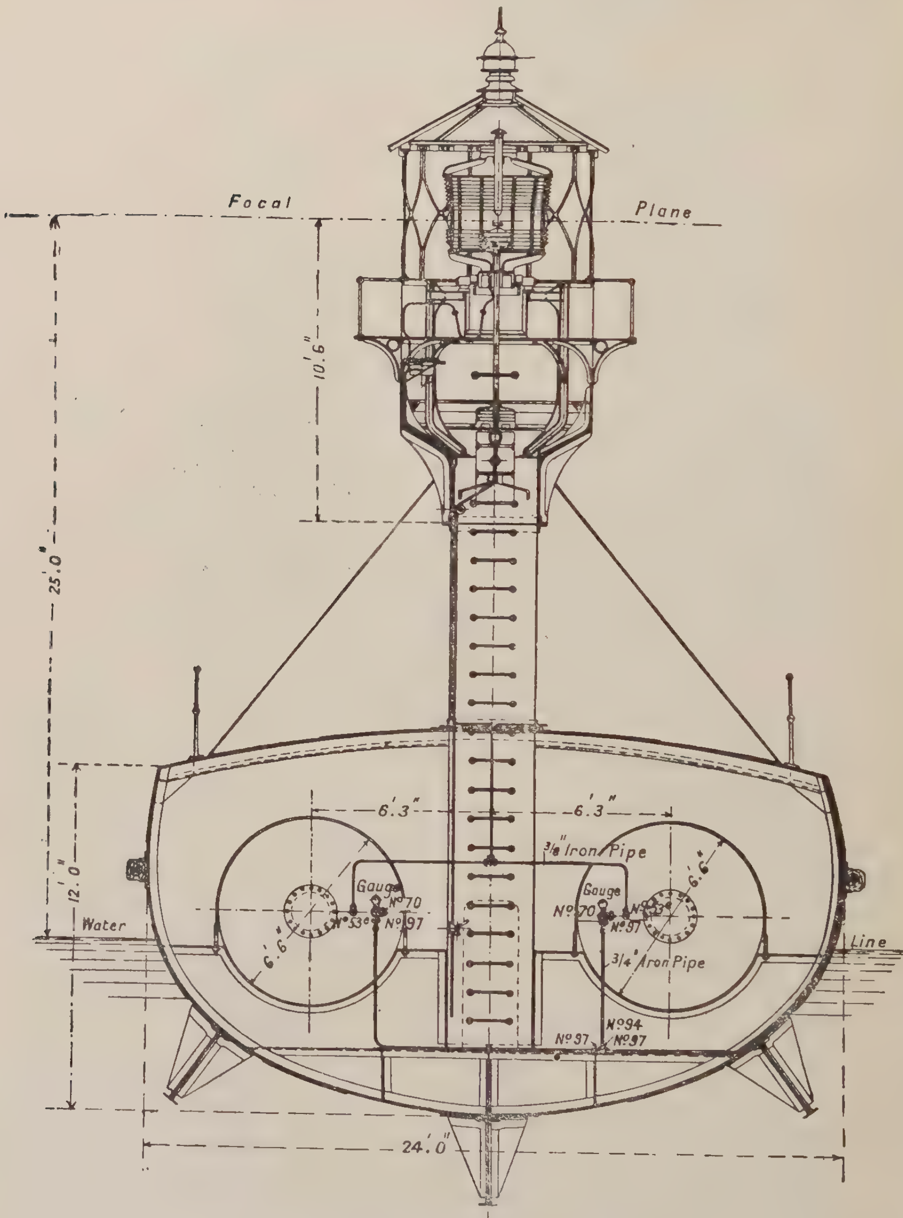


Fig. 275.—Section of Otter Rock Lightship.

tubes, or bottles, of about 27 litres capacity, of which only two-thirds is occupied, the remaining one-third volume being left as a margin of

[*To face p.* 338.

Fig. 276.—Otter Rock Lightship.

safety for expansion. The gas cannot be used direct from the tubes, but has to be expanded under suitable precautions into an intermediate receiver, to a pressure of 6 to 10 atmospheres. The process of making Blau gas is much more expensive than that of making ordinary oil gas; the compensating advantage claimed for it is that, on account of its liquefaction, the gas can be more readily and conveniently transported from the place of manufacture to the buoy. It is also richer in heavy hydrocarbons, and its illuminating power is correspondingly higher. With an open flame, oil gas affords ·2 Hefner candle unit per litre per hour and Blau gas ·4 Hefner unit. These may be compared with $1\frac{1}{2}$ to $1\frac{3}{4}$ Hefner candle power produced by acetylene.

Acetylene can be readily obtained either by the admission of water to carbide, or by the reverse process of allowing granular carbide to fall into water, the quantities being suitably regulated in each case. Both these systems have been applied to buoy lighting, and they are carried out *in situ*, the gas being produced as required. For an open flame the result, so far as the light is concerned, has proved quite satisfactory, but in burning with incandescent mantles difficulties have been experienced owing to impurities in commercial carbide: the phosphites producing deposits of phosphoretted hydrogen. This defect is remedied in the case of Dissolved Acetylene, the gas being thoroughly purified and washed before compression into the steel cylinders in which it is stored.

Dissolved Acetylene.—The compression of acetylene for storage purposes is, as already stated, generally illegal, on account of the liability of the gas, so stored, to explode. On the other hand, compressed acetylene in what is known as the dissolved form is perfectly safe, and cylinders of such acetylene, prepared in accordance with official regulations, are classed by the Home Office as non-explosive.

The steel cylinders [1] for the reception of the dissolved acetylene are first completely filled with a substance having a porosity of 80 per cent., which is then caused to absorb as much acetone as would occupy 40 per cent. of the volume of the cylinder. The actual gas capacity of cylinder is thus 40 per cent. of 25, or 10 volumes per atmosphere of pressure.

Dissolved Acetylene cylinders should not be discharged below 1 atmosphere gauge pressure, because the rate at which the acetone is carried off by the gas greatly increases as the pressure approaches zero. Acetone vapour has no detrimental effect upon the consumption of the gas in the burner. It is generally found necessary to make up the quantity of acetone, after five to ten refillings of the cylinder, with acetylene.

Economies in the consumption of dissolved acetylene have been effected by the introduction of mechanism for emitting flashing or occulating lights, whereby a wide range of intermittent light signals can be produced automatically.

Another contrivance is the *Sun valve*, or, more correctly the *Light valve*,

[1] The receptacles for dissolved acetylene are generally pressed, not welded.

which is a mechanical device sensitive to light, and actuated entirely by the alternation of daylight and darkness. It opens the gas supply at nightfall and closes it at daybreak with perfect regularity, resulting in a saving of some 25 to 40 per cent. of the gas, according to climate, as compared with a constantly burning light. The use of the instrument is generally restricted to shore lights, or to lights of considerable magnitude.

Mr. Dalèn, a Swedish inventor, has produced an automatic apparatus for mixing in correct and constant proportions the dissolved acetylene and air, and for burning the compound in conjunction with incandescent mantles, either as a constant or intermittent light. This apparatus, also, by its nature, is most applicable to lights of considerable power, superior to those generally fitted to buoys.

As an instance of the cost of buoy illumination by dissolved acetylene may be mentioned nine light buoys in the entrance channel to the Port of Lido, Venice, which command a light range of 6 miles under conditions of average atmospheric clearness, and contain a supply of gas for a period of 110 days. The lenses are 300 mm. diameter, and the burner consumes 15 litres per hour. According to a statement of Sig. Carlo Duse in the *Giornale del Genio Civile*, the annual cost per buoy, covering attendance and repairs, and four charges of gas, each of three cylinders of 5 cubic metres, at 4s. per cubic metre, is £15 12s.

The Swedish Lighthouse Department, which has adopted the A.G.A. system of lighting covering the patents of Mr. Gustaf Dalèn, has in the majority of cases obtained good results from a light with flashes lasting $\frac{3}{10}$ second and recurring every three seconds. The saving of gas amounted to 89 per cent. The sun valve has in several cases been introduced in England, but this, while found to work satisfactorily in lighthouses, has not yet been applied to buoys.

Attachment of Illuminating Apparatus to Buoys.—Except in the case of Wigham lamps, no special contrivance is found necessary to ensure the verticality of the illuminating apparatus of buoys in disturbed waters. Ordinary fixed attachments are made to serve. The principle of securing the Wigham lamp by gimbals will be explained in connection with lightships.

Lightships.—Buoys are not invulnerable, and it is quite within the bounds of possibility that a luminous buoy may be extinguished from one cause or another, though the occurrence is by no means as common as might be supposed. Where there are heavy seas and strong currents, however, lighted buoys are exposed to undue risks, and signals of a more reliable character are desirable. Lightships are much steadier under these conditions: their oscillation is less, and they are not so liable to be put out of action. If a buoy should happen to be run down in a fog there is some natural, though reprehensible, disposition on the part of the navigator responsible, to shirk his liability by omitting to report the accident, and the omission has had serious consequences. A light vessel, on the other hand, even if unattended

by a crew, is too prominent a feature to be ignored in this manner. Another point is that light vessels enable the focal plane to be set at a considerably

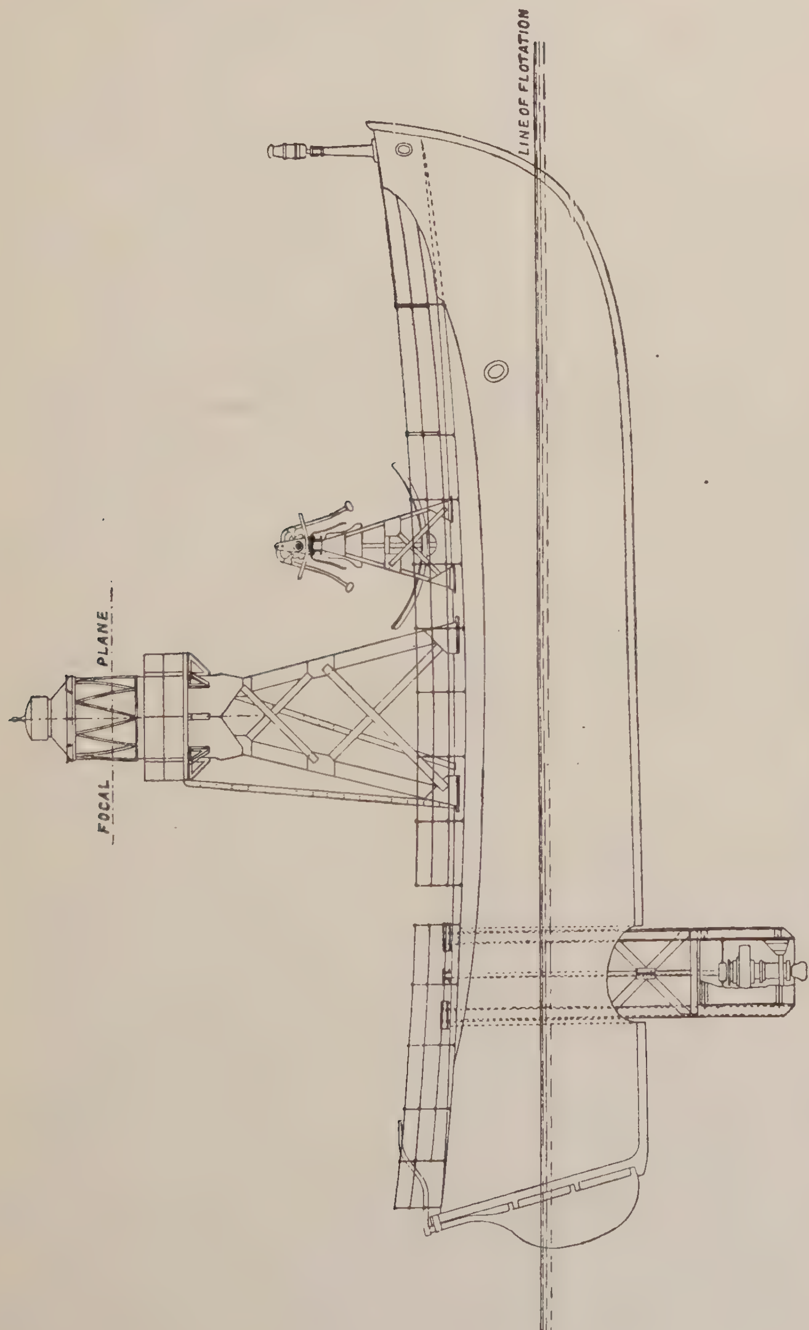


Fig. 277.—Unmanned Lightship (Trinity House Type).

higher and more effective level, and an increased intensity of illumination is obtainable.

Steadiness is one of the most essential qualities of a lightship, and the attainment of it, so far as such a thing can be realised amid unstable surroundings, involves the suppression of synchronism in the periods of oscillation, respectively, of the vessel and of the waves. Synchronism is more likely to occur transversely under the action of rolling, than longitudinally, under the action of pitching. In order to avoid it, ballast should be located at some distance from the centre of gravity of the vessel, so that the moment of inertia of the latter about its longitudinal axis may be as great as possible and the metacentric arm reduced to the minimum consistent with stability.

A metacentric height of 14 or 15 inches has been indicated as the correct value for lightships generally, but this standard has lately been questioned by Mr. George Idle, M.I.N.A., the Naval Architect to the Commissioners of Irish Lights. In a paper read before the Institution of Naval Architects he suggests that the metacentric height is not the pronounced factor it appears to be.[1] Several vessels designed by Mr. Idle have metacentric heights in excess of 15 inches, the required steadiness being obtained by the attachment of very large bilge keels. There is no doubt that the steadiness of lightships is promoted by deep central and markedly protruding bilge keels, to all of which ballast may advantageously be affixed. In some of the latest examples of lightships, the keels have a depth exceeding 3 feet.

The following observations by Mr. Idle on the behaviour of Irish lightships at their stations are of interest :—

"1. The greatest rolling amplitudes are attained by old ships of the wooden or composite class, having the double bilge logs, and which have, generally, small initial stability and a low metacentre. In other words, the G.M. does not by itself give any indication of the ship's probable behaviour in a heavy sea.

"2. Large amplitudes are reached when the sea is 'breaking' (therefore steep), or, as it is termed, 'confused,' and when the waves are advancing on the bows or quarters. Maximum amplitudes have, in fact, been recorded when the ship has been nearly 'head to' the advancing wave.

"The differences between the amplitudes obtained during the above-mentioned circumstances, and those that are obtained when the ship is riding exactly 'beam to' a heavy swell, is very marked, averaging 16° to 20° for the single oscillation in favour of the 'beam to' position. This suggests that there are causes productive of heavy rolling other than mere assonance between the ship and the wave.

"3. The greatest angle of heel in a bad sea is always to the 'lee side'—that is, 'away' from the advancing wave—no matter what the direction and force of the wind may be.

[1] Idle and Baker on the Effect of Bilge Keels on the Rolling of Lightships, *Proc. Inst. N.A.*, March, 1912.

"4. Where the bilge keels are efficient, the ship's normal period of oscillation is increased by 1 up to 3 seconds, and sometimes more. When the ship is rolling in this increased period her amplitudes of roll are generally moderate. There is, of course, between the actual period of the ship and that of the wave a certain correspondence, but no continuous synchronism, even when the normal period of the ship and the period of the wave are identical. All that can be definitely said on this point is that there is apparently an attempt on the part of the wave to bring the ship to its own period. Here is seen the chief function of the bilge keel. It prevents assonance between the ship and the wave. Indeed, it may be safely asserted that, without bilge keels, inefficient as they may be in some cases, these small vessels could hardly live in the seas they are sometimes exposed to."

The foregoing statements, it must be understood, refer particularly to *maximum* amplitudes of roll, which are the chief concern as affecting the maintenance of the lighting and timing apparatus in perfect order and regularity.

The dimensions of lightships have materially increased of recent years. A length of 60 or 70 feet used to be fairly general, but now a number of vessels have lengths well over 100 feet, while others range up to 150 feet. The depth and draught of these vessels manifest a proportionate increase, but the beam has, if anything, tended to diminish, or, at least, to remain stationary for some little time. The accompanying table classifies the leading dimensions of certain representative vessels of all three nationalities.

Dimensions of Representative Lightships of Various Nationalities.

Nationality.	Name or Locality.	Length.	Beam.	Depth.	Draught.	Height of Focal Plane above Water.
		Feet.	Feet.	Feet.	Feet.	Feet.
British, .	Longsand, . .	60	24	12	6¾	30
French, .	Talais, . . .	61	20	9	6¼	33
,, .	Snouw, . . .	65½	20	13	11¾	33
British, .	Gaspar Point, R. Hooghly, .	75	28	12	6¾	35
,, .	R. Mersey, . .	103¼	21¼	11	9½	30
French, .	Sandettié, . .	115	20½	16¾	15	39¼
British, .	R. Mersey, . .	118¾	21	11¼	9½	30
German, .	Fehrmarnbelt, .	134½	24¼	17	..	..
British, .	Trinity House, .	104	26	15	10½	35
,, .	Mersey Bar, . .	104	24	15	10¾	32
Dutch, . .	Noord Hinder, .	138½	23¼	13	10½	..
French, .	Grand Banc, . .	65½	19½	10½	8¾	39⅓
,, .	Rochebonne, . .	47½	21½	9	7½	33
,, .	Havre, . . .	131	19½	16¾	15	..
German, .	Amrumbank, . .	150	26¼	18	14	..

The Focal Plane, which is the horizontal plane through the centre of the optical apparatus, is generally fixed at some height between 30 and 40 feet above the sea level.

Suspension of Floating-lights.—In order to maintain verticality, the illuminating apparatus of a lightship is supported on gimbals. In catoptric [1] lights, the reflectors and lamps are suspended in this manner from above. With the dioptric system,[1] the apparatus is generally hung in the form of a pendulum swinging about a horizontal axis located immediately beneath the lamp. The pendulum, or rod, is weighted and counterweighted above and below, the weights being adjusted in such a way that the period of oscillation of the lamp is considerably longer than that of the vessel, so that the maximum inclination of the former may not exceed 5 or 6 degrees. Manifestly, the apparatus must not only be sufficiently sensitive to maintain its verticality, but it must also admit of free and ready response to change of direction, and this is secured by attaching the gimbals to a horizontal circle rotating on steel balls. In the event of exceptionally heavy rolling on the part of the vessel, the possibility of collision between the pendulum and the lower part of the lantern may be guarded against by the provision of a thick annular pad of india-rubber on the weighted portion of the latter, or by restricting the swing of the pendulum with the aid of check chains and flexible guys.

Lightship on Mersey Bar.—The following, condensed from a paper by Lieut. Gracey, R.N.R.,[2] is a brief description of the lightship "Alarm," stationed on the Mersey Bar, and built, in 1912, by Messrs. Hawthorns & Co., of Leith, to the order of the Mersey Docks and Harbour Board.

The length between perpendiculars on the load-line is 104 feet, the breadth (moulded) 24 feet, the depth (moulded) 15 feet, the draught forward 10 feet 7 inches, and the draught aft 10 feet 10 inches.

The shell below the sheer strake is of BB iron plates, and elsewhere of Siemens-Martin steel; generally, its thickness is from $\frac{7}{16}$ inch to $\frac{9}{16}$ inch. The centre keel is of BB bar iron, 9 inches by $2\frac{1}{8}$ inches, and there are bilge keels for three-fifths of the length amidships, composed of iron bulb plate, 11 inches by $\frac{10}{16}$ inch, double rivetted to a **T**-bar 6 inches by 5 inches by $\frac{10}{16}$ inch. The frames are of steel angle bulb, $5\frac{1}{2}$ inches by 3 inches by $\frac{6}{16}$ inch, spaced 20 inches apart, and the floors are 18 inches deep by $\frac{7}{16}$ inch. The keelson, sister keelsons, and side stringers are all of heavy section steel bulb angles. The beams are of Butterly bulb steel, 6 inches by $4\frac{1}{2}$ inches by $\frac{7}{16}$ inch, and are fitted in the main and lower deck on alternate frames. The main deck is of teak, 5 inches by $3\frac{1}{2}$ inches, and the lower deck of red pine, 7 inches by $2\frac{1}{2}$ inches. There are six water-tight bulkheads of $\frac{5}{16}$-inch steel plating, kneed to the side and lower deck stringers, and stiffened by angles, 3 inches by 3 inches by $\frac{7}{16}$ inch, spaced 2 feet 3 inches apart. The bulkheads

[1] See p. 356.

[2] Gracey, on "The New Mersey Bar Lightship," *Min. Proc. L.E.S.*, vol. xxxv.

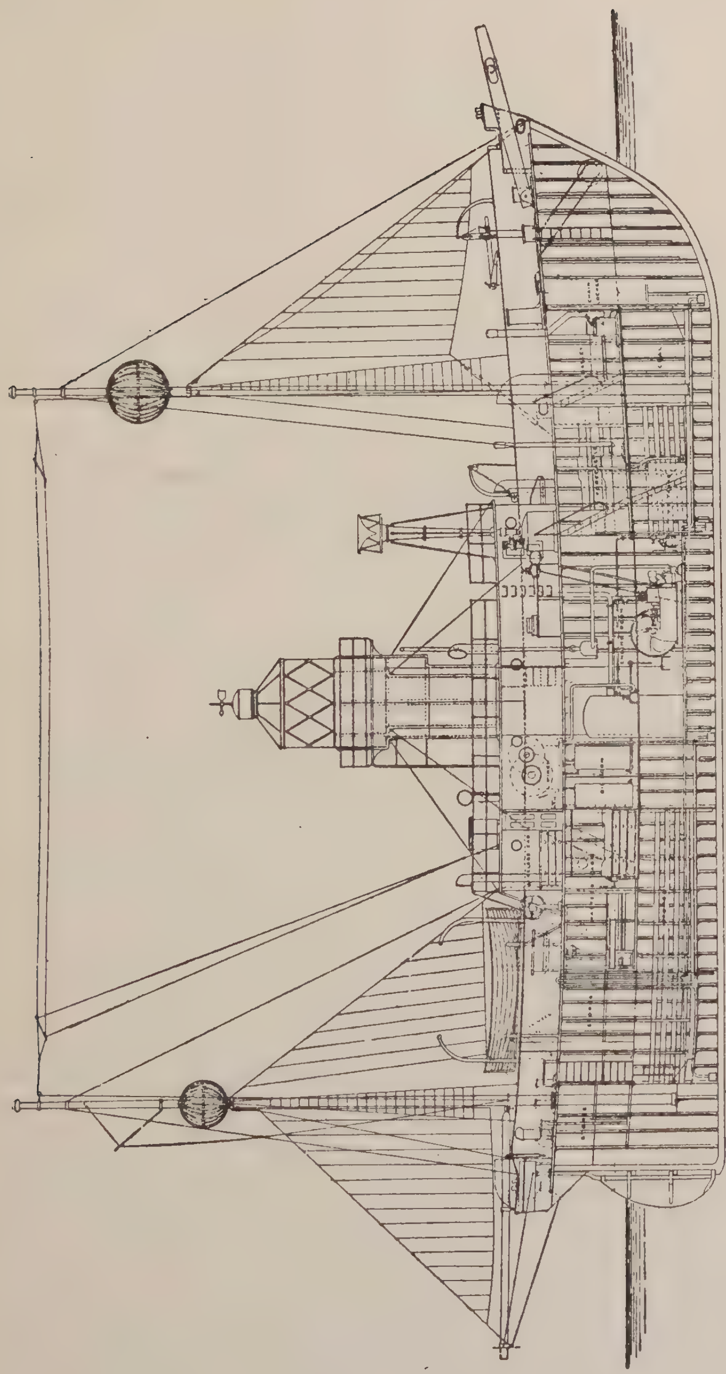


Fig. 278.—Lightship "Alarm" (Mersey Docks and Harbour Board).

are so arranged that the ship will remain afloat with any two adjoining compartments full of water.

There are two steel masts, with the requisite height and spread for the aerial of a wireless telegraph installation, which is of 200 watts capacity, and has a working range of about 20 miles. The masts also carry the day signal of two red balls, and are used for visual signalling.

The main moorings consist of $1\frac{3}{4}$-inch long link chain cable, subjected to a proof strain of $37\frac{1}{2}$ tons, the breaking strain being, at least, double this amount. The anchors weigh 40 cwts. each, and are of Rodger's improved box stock type. The moorings are arranged in two legs, run out in opposite directions, on a line about N.W. by W., as it is from this quarter that the majority of the gales blow and the heaviest seas arise. The north-west leg has a length of 165 fathoms, and the south-east leg of 120 fathoms. The mooring swivel has a breaking strain of about 150 tons. It is slung from the main hawse pipes by two 8-fathom bridles of special $2\frac{1}{16}$-inch short link chain, the lower ends being shackled to the links in the top of the swivel. These bridles are made fast by taking five round turns with each on two single mooring bollards, fitted on the deck about 8 feet abaft the top end of each hawse pipe. The two ends of the bridles are lashed together on the after side of the bollards, and, in case of emergency, can be easily slipped by one man. The top ends of the two legs of mooring chain are shackled to the links in the bottom of the mooring swivel, sufficient slack being left to enable the vessel to ride easily in heavy weather.

The winch for handling the moorings is placed well aft, and is driven by compressed air at a pressure of 40 lbs. per square inch, supplied by the fog-siren plant.

The spare moorings consist of 200 fathoms of $1\frac{1}{4}$-inch studded chain cable, divided into two legs for the port and starboard sides, and the anchors, which weigh 17 cwts. each, are stowed on sloping billboards on each bow, and fitted with slip-releasing gear.

The lantern tower is circular in plan, 26 feet 6 inches high by 7 feet diameter, formed of $\frac{3}{8}$-inch steel plates strengthened by eight vertical bulb tees extending the full height, and also by two floors, supported by horizontal bulb tees, one at the base of the lantern and the other at the level of the main deck. The base of the tower rests on a special steel bed on the top of the floor plates, and is secured thereto by a ring of angle steel, 5 inches by 5 inches by $\frac{3}{4}$ inch. It is fitted with a similar ring at the level of the main deck, but passes through the deck-house top with a clearance of 2 inches all round, to allow for any springing tendency in bad weather. Eight flexible steel $4\frac{1}{2}$-inch wire stays are fitted at equal intervals to the tower as preventer supports.

The lantern is of steel, 9 feet in external diameter, tapering in the lower part to 7 feet, and is glazed with $\frac{3}{8}$-inch plate glass. The dome of the lantern stands 48 feet above the keelson.

The optical apparatus, supplied by Messrs. Chance, of Birmingham, is of the small third order size, lighted on the incandescent vaporised petroleum system. It has a focal distance of 375 mm., and comprises three

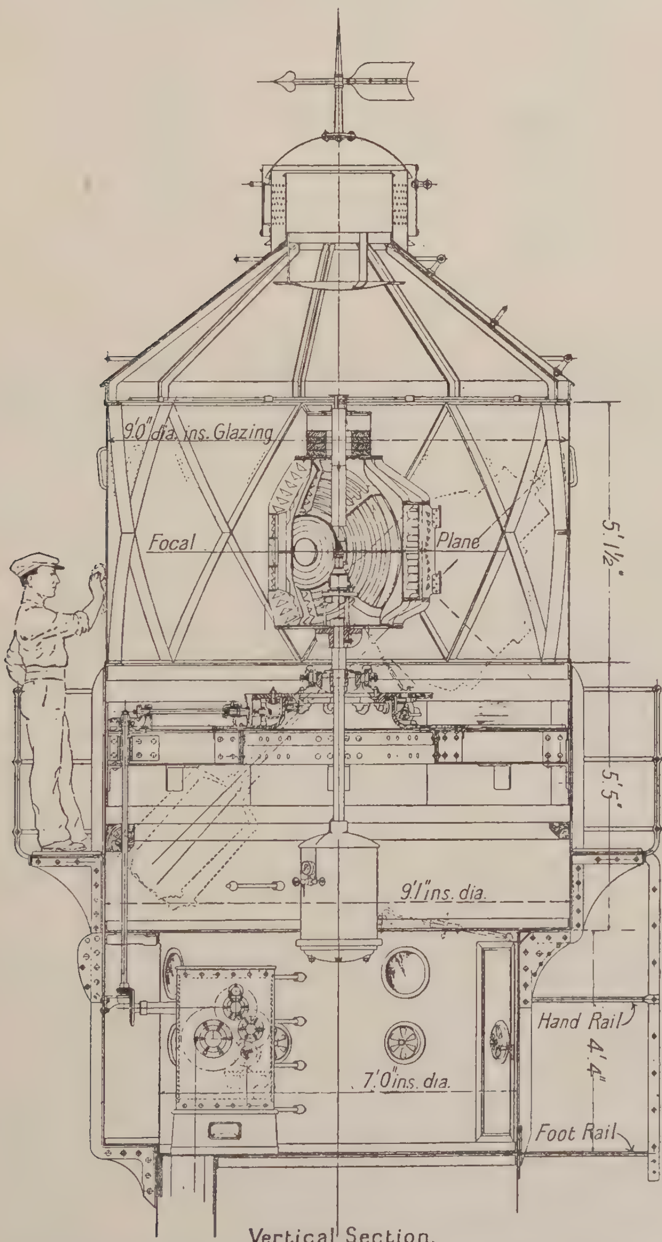


Fig. 279.—Triple Flashing Light and Lantern in Lightship "Alarm."

panels of refracting and reflecting prisms. Two of the panels subtend a horizontal angle of 99° each, and the third an angle of 81°.

There is also a dioptric mirror of 81° horizontal angle and 500 mm. focal distance, placed directly opposite the central flashing panel for the purpose of strengthening it. The apparatus has a triple flashing characteristic, as follows :—

Flash,	·58	seconds.
Eclipse,	4·08	"
Flash,	·58	"
Eclipse,	4·08	"
Flash,	·58	"
Eclipse,	20·10	"
Total,	30·00	"

The pendulum, which carries the lenses and burner, is supported by a steel table carried by a gimbal ball bearing, which itself is supported by a carriage fitted with horizontal and vertical rollers, all supplied with ball bearings. The gimbal carriage and pendulum are designed for a period of 11 seconds and for a swing of 45° on either side of the vertical, the amount of actual swing being partially governed by movable counterpoise weights.[1] A circular wooden fender is fitted inside the lantern, and the base of the pendulum is covered with a heavy rubber buffer to take the shock in case of contact. The top table of the carriage is provided with a gun-metal race wheel, which is actuated by means of a bevel pinion in connection with horizontal and vertical shafting from the clockwork mechanism. The clock is placed on the lantern floor, and is driven by a falling weight, fitted with sheaves on each face for travelling in the guide frames down the inside of the tower.

The lower part of the pendulum contains the receptacle for the oil and compressed air. One filling of oil is sufficient for the longest night, and the pressure is supplied either by a hand pump, or direct from the main storage receivers.

The incandescent burner at the top of the pendulum carries a mantle of 55 mm. diameter, and produces a beam of approximately 35,000 candle-power through the lens, with a vertical divergence of 6° above and below the horizontal.

The height of the focal plane above sea level is 32 feet, and the range in clear weather is 11 miles, but the "loom" of the light has been observed at a distance of 20 miles.

The stand-by auxiliary illuminant consists of a powerful two-wick capillary

[1] Under the quick movement of the ship due to some sudden swell or an abnormal wave, occasionally, at first, the pendulum struck the side of the lantern tower. This tendency was rectified by a process of trial and error by an adjustment of the weights.

lamp, which can be fitted in a few minutes, and is stowed in a carrier close at hand.

The approximate weight of the tower is 11 tons; of the lantern, 6 tons 3 cwts.; and of the lamp, pendulum, carriage, and clock, 2 tons 1 cwt., making a total of nearly 20 tons.

The vessel is also equipped with a fog-siren apparatus designed to give a blast of 2 seconds' duration every 20 seconds, and with apparatus for the transmission of submarine signals, particulars of which are given elsewhere.[1]

Lightship Attendance.—The reliability and automatic continuity of the compressed oil gas illuminating apparatus has very largely done away with the necessity for crews on board lightships. In many cases now these vessels are unattended, and only visited at long intervals for the purpose of supplying fresh gas. This has effected considerable economy in maintenance expenses, and extended the scope of utility.[2]

The liability, however, of all floating objects to displacement, is the inherent weakness of the lightship, as also of the light-buoy. A displaced signal is much worse than none at all. Beacons and lighthouses, therefore, from their very fixity, possess uncontrovertible merits as regards accuracy of alignment, and it is usual to rely mainly upon them in so far as they happen to be available for this purpose.

Lighthouses and Luminous Beacons.—The earliest type of the lighthouse was the lighted beacon, usually situated upon a natural eminence or upon a tower. It was an iron-barred grate, or receptacle, for wood and coal, which was ignited at night-time. These signals were, therefore, most crude and primitive, and often gave out more smoke than light. Moreover, they lent themselves to easy reproduction and imitation for illicit ends. In this form they have long since disappeared into the lumber of the past. Their present development is the harbour light, a lantern attached to the top of an upright mast, which is used at the entrances of minor ports.

The lighthouse is a tall structure, occasionally of wood, but much more commonly of stone, reinforced concrete, or iron, rising oftentimes to a considerable height above the water level. When, however, a natural headland or cliff lends itself to the purpose, the structure is not necessarily lofty, and, indeed, for channel lighting, no great height is essential. The building is usually planned in a series of stages or floors, the lantern containing the illuminating apparatus being located at the summit.

The practice of channel lighting, developing through luminous buoys to lightships, attains its highest degree of utility and perfection in the lighthouse. Sources of illumination for lighthouse use are not only numerically and potentially greater than those available for buoyage service, but they are also of an even more diverse nature, including electricity, coal gas,

[1] *Vide* p. 364.

[2] The lightships "Rochebonne," "Snouw," and "Grand Banc" in the French Service, particulars of which are given in the table on p. 343, are without crews.

mineral, vegetable and animal oils, oil gas, and acetylene in various forms. The feeble and ineffective candle, which maintained its footing for at least thirty years of the last century, has now entirely disappeared. Its great modern prototype is the electric arc, the crater of which possesses an intrinsic radiance of over 55,000 candle-power per square inch of illuminating surface. Beams of light can now be projected far beyond the limits of their geographical range. The mariner sees their reflection in the sky before he comes within direct visual contact with them. This, of course, applies to landfall lights, and not to the class of lights which form the subjective basis of this chapter. Channel lighting is achieved perfectly satisfactorily with the aid of lights of a far lower calibre. The electric light is rarely, if ever, employed for this purpose. For general use, the incandescent petroleum vapour burner is eminently suitable, convenient, and much less costly, and its light is sufficiently powerful for all stations other than those of primary and special importance. The system was indeed, only introduced into this country at the beginning of the present century, but it had been adopted in the French lighthouse service several years previously, and it may now be said to have attained general recognition. Prior to its introduction the wick burner was practically universal, either flat, as in the earlier instances, or cylindrical on the Argand principle, with as many as six, eight, or ten wicks arranged in concentric rings. Despite the inferiority of the wick apparatus to the electric arc, its illuminative power was a very long way ahead of the feeble glimmer emitted by the cluster of tallow candles which lit up the summit of Eddystone a century ago. Twenty-four of these candles unitedly gave a light equivalent to sixty-seven standard candles.[1] In the later Eddy-

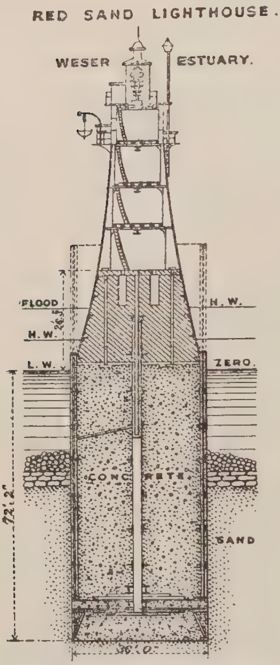


Fig. 280.—Red Sand Lighthouse, River Weser Estuary.

[1] For a definite comparison of the various illuminating agents, several standards are in use. The legal standard in this country is the spermaceti candle burning 120 grains of spermaceti per hour, but, as a practical primary standard, the 10-candle Harcourt pentane lamp is now generally employed, as it has been carefully adjusted to have a candle-power ten times that of the legal spermaceti candle. In France, the Carcel lamp is the standard chiefly used. It burns 42 grammes of pure Colza oil per hour, and is equivalent to 9·8 English candles, 9·6 French candles, and 8·6 German candles. A provisional International candle has, however, been agreed between the British, French, and American authorities. It has a luminous intensity equal to one-tenth of the Harcourt pentane 10-candle lamp. Germany continues to retain the Hefner (amylacetate) standard lamp, the agreed value of which is 0·9 of the so-called International candle. One such International candle is equal, therefore, to 1·11 Hefner units.

stone of 1882, Douglass burners, with six concentric wicks, attained an aggregate of nearly 80,000 candle-power. With the incandescent oil burners now in use, the resultant beam is of 292,500 candle-power.

But the true standard of comparison is not so much the gross illumination as the intensity per unit of area. It is this intrinsic intensity which confers upon a beam its penetrative power. The brightness of the flame section on the old wick burners ranged up to 70, or, at most, 80 candle-power per square inch, according to the number of the wicks. With incandescent mantles there is, generally speaking, an increase in intensity of 300 per cent., equivalent to about 200 to 250 candle-power, while at the same time the consumption of oil is reduced by nearly one-half.

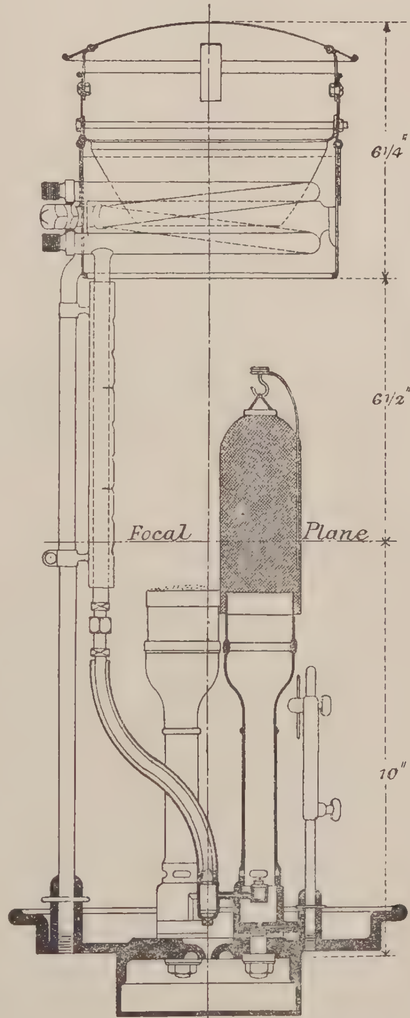


Fig. 281.—Matthews' Triple Mantle Burner.

Incandescent burners are numerous, as well as varied, but they fall naturally into two main types or classes. One class is that in which vaporisation is effected by the direct heat of the mantle, and for this purpose the vaporising chamber, or tube, is placed close to or above the mantle. In the other class, a separate heater is used for vaporisation. Obviously, this entails additional apparatus; but, on the other hand, there is less interference with the luminous range of the mantle. In all cases, some temporary source of heat is required for a preliminary five or ten minutes, until the action of the burner becomes automatic.

Burners of the first class include the **Matthews** burner, the **Luchaire** burner, and the **Pintsch** burner. The first two have upright mantles; in the third case the mantle is of the inverted type.

In the **Matthews** burner (fig. 281), vaporisation is effected in a brass tube coiled above the mantle and inclosed in a metal hood.

In the **Luchaire** burner, type *a* (fig. 282), the vaporising tube takes the form of the letter U inverted, and arches the mantle. In the alternative form, type *b* (fig. 283), it projects upward in a curve on one side only.

Manifestly, the piping necessary to convey the oil to and from the source of vaporisation obstructs the rays emanating from certain sections of the

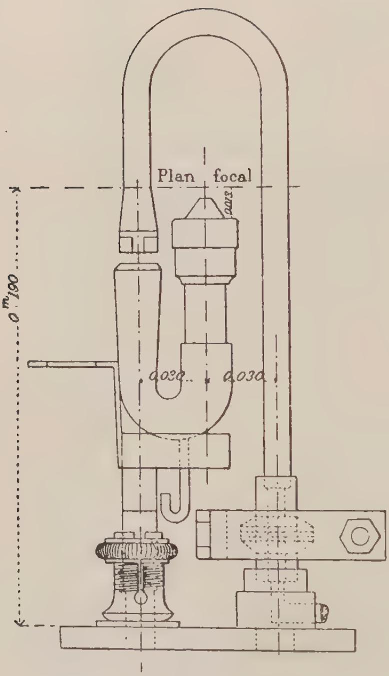


Fig. 282.—Luchaire Burner, Type *a*.

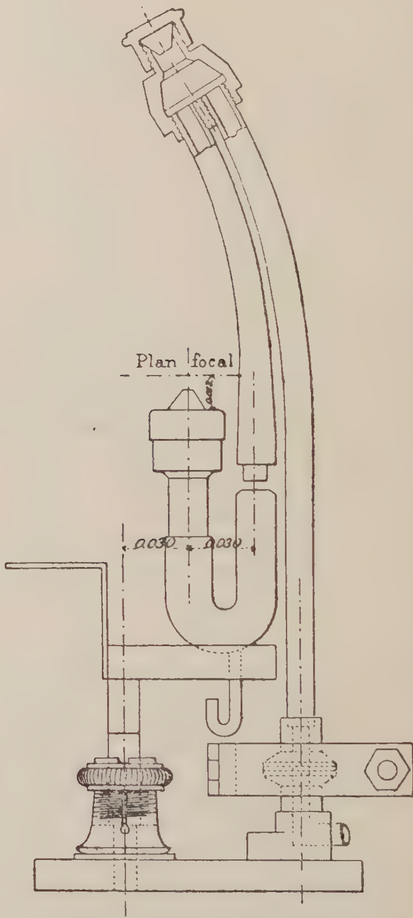


Fig. 283.—Luchaire Burner, Type *b*.

mantle. Moreover, the bends in the piping make it difficult to keep a clear passage for the vapour.

In the **Pintsch** burner (fig. 284) with inverted mantle, the oil is vaporised while descending the spiral groove of a central vertical duct.

Burners of the second type include the **Chance** burner, the **Scott** burner, and the **Pintsch** burner (upright mantle).

In the **Chance** burner the dual horizontal vaporising tubes are contained in a metal chamber below the burner proper, and they are heated by a

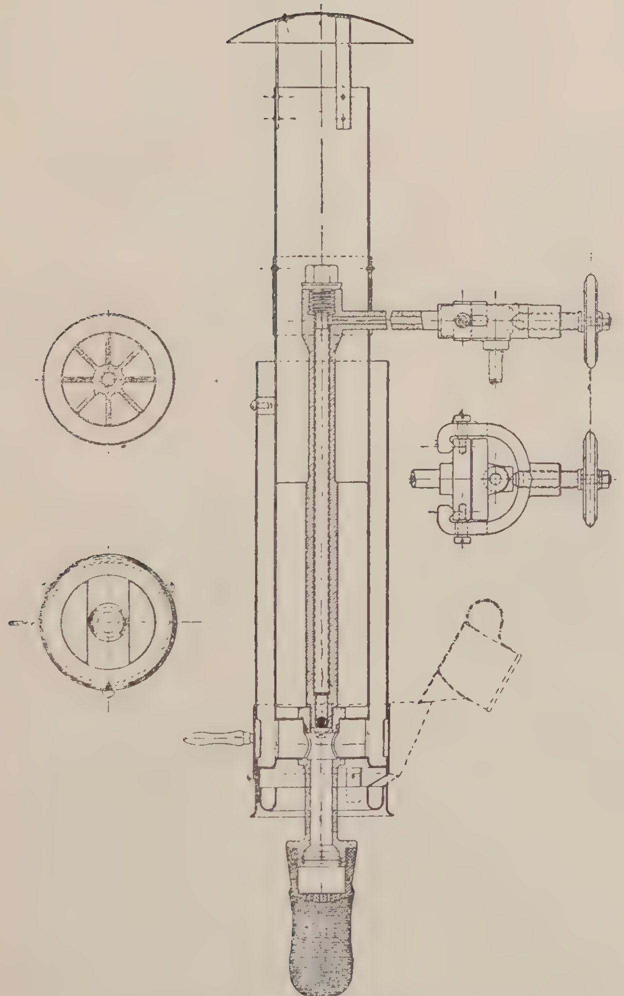


Fig. 284.—Pintsch Burner with Inverted Mantle.

subsidiary burner, deriving its gaseous supply from the products of vaporisation.

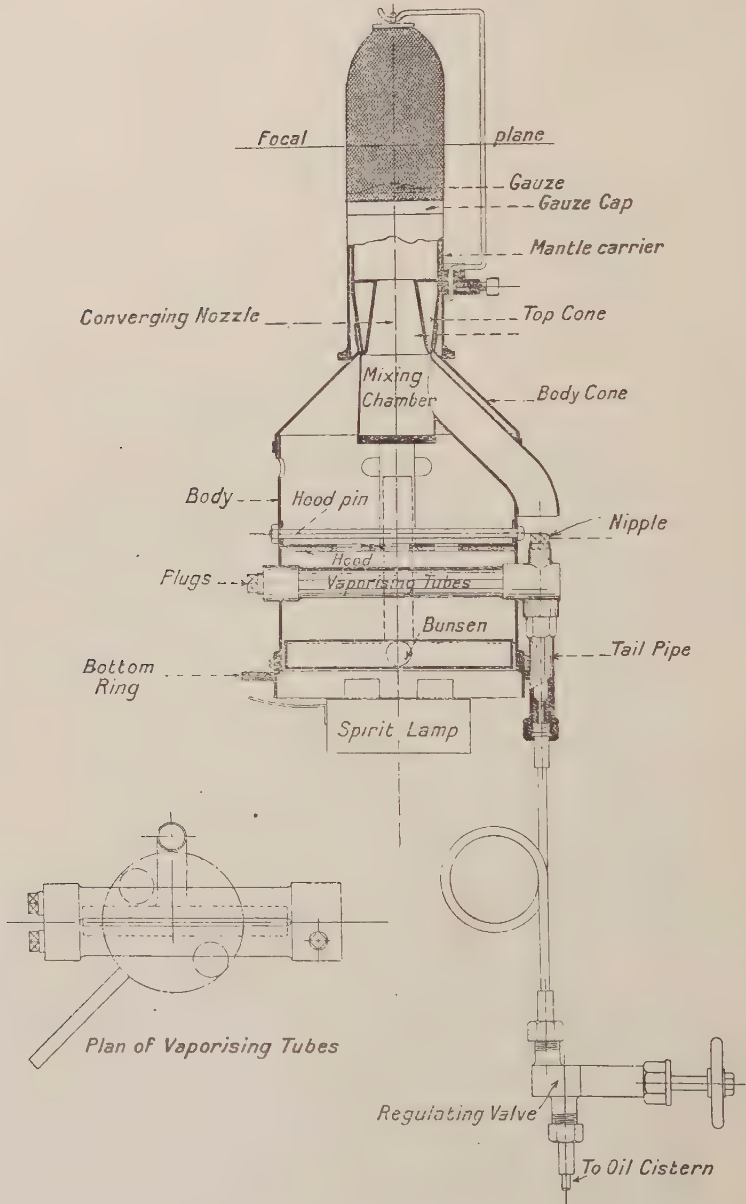


Fig. 285.—Chance Incandescent Burner.

In the **Scott** burner (fig. 286), a series of subsidiary burners, inclosed in a metal cover, heat the conducting tube in a vertical position. These

subsidiary burners likewise draw their alimentation by means of a bye-pass from the mixing chamber.

In the **Pintsch** burner (fig. 287) of the second class a portion of the vaporised oil rising in the central tube is deflected by a baffle plate, set immediately below the mantle, and caused to pass down the arched side tubes, the lower portions of which form the subsidiary burner.

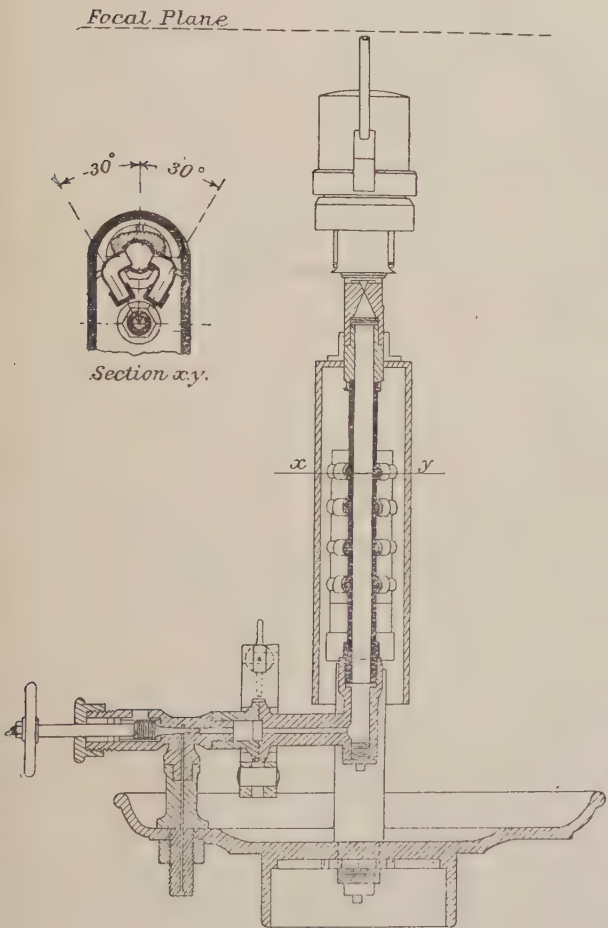


Fig. 286.—Scott Burner.

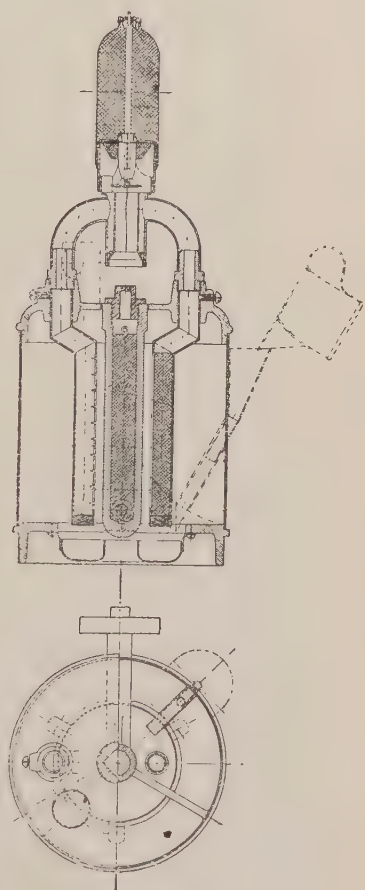


Fig. 287.—Pintsch Burner.

Acetylene.—The use of acetylene has already been referred to in connection with buoy lighting (p. 339), and it is only necessary to add that the facilities for its use as a fixed light, either in the simple or concentrated form, are greater than in the case of buoys. The danger of explosion is perhaps somewhat more pronounced on account of the confined space in a lighthouse tower or lantern, and special precautions are desirable where

acetylene is used otherwise than in the dissolved form. Owing to reasons of cost, which is much greater than that of incandescent oil, the use of acetylene is not general for large lights, but it appears to be well adapted for less important and unattended positions.[1]

The types of burners used with acetylene are numerous; they are generally arranged in clusters varying with the luminous intensity required. Acetylene burners are also constructed for use with the incandescent mantle, and this reduces the cost per candle-power, but does not bring it down to the level of incandescent oil. In the Dalèn light a special automatic aspirator or mixer is introduced for the purpose of combining the requisite quantity of air with the gas in order to secure perfect combustion. There is also a contrivance which automatically withdraws a new mantle from a magazine in the lantern and places it on the burner each time a mantle in service breaks.

Light Concentration.—The source of light being but one of the factors in the determination of lighthouse efficiency, we now turn our attention to the methods adopted for the concentration and intensification of the issuing rays.

The simple, undirected flame is wasteful of light—that is, much of the light is lost to useful purposes. Most lighthouses stand upon the coast-line, and the area of radiation, therefore, frequently includes a large sector of land over which illumination is entirely unnecessary. Also, apart from this cause, a good deal of light is lost by diffusion and dispersion.

To remedy these defects, reflectors were introduced as far back as the latter half of the eighteenth century. At first spherical in form, the mirror ultimately became parabolic, concentrating the emergent rays of the light from the focus along a path parallel to the horizontal axis. This constitutes the **catoptric** principle. Catoptric reflectors are of two types: first, the paraboloid, formed by the generation of a parabola about its own axis, and sending the light rays in a single direction only; and secondly, the dual (upper and lower) surfaces formed by the horizontal rotation of a parabola round a vertical axis through the focus. This arrangement, while confining the light within vertical limits, distributes it equally throughout a horizontal plane. It is not, however, practised in harbour work, and the former is the type alone in vogue. Except in lightships where there is still some risk of the breakage of glass lenses from the motion of the vessel, the use of catoptric reflectors is obsolescent, and with the improvements now being effected in balancing and other respects, the tendency is for the application of the system to become more and more restricted.

The **dioptric,** or **lenticular,** principle of ray concentration, based on the refractive properties of lenses, is due to Augustine Fresnel, who initiated it,

[1] For instance in his Report to the Twelfth International Congress of Navigation, Mr. Grönvall, Chief of the Swedish Lighthouse Department, stated that with the introduction of the A.G.A. (dissolved acetylene) light, the staff formerly required for eleven lighthouses along the principal passage to the port of Stockholm had been dispensed with.—Grönvall on Security of Navigation, *Twelfth Int. Nav. Cong. Philadelphia,* 1912.

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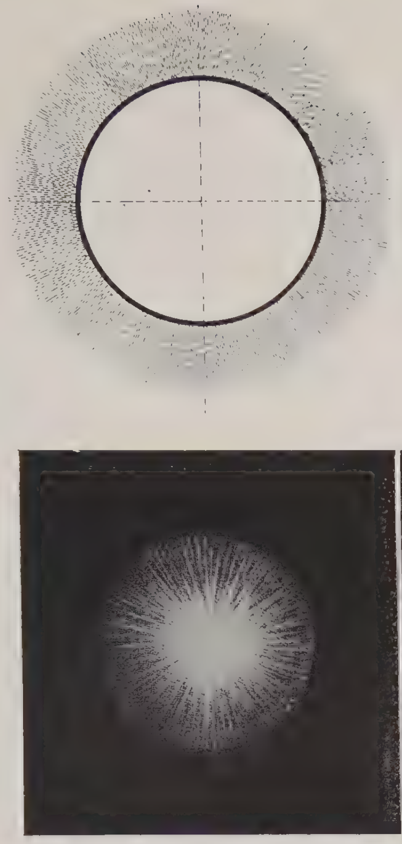
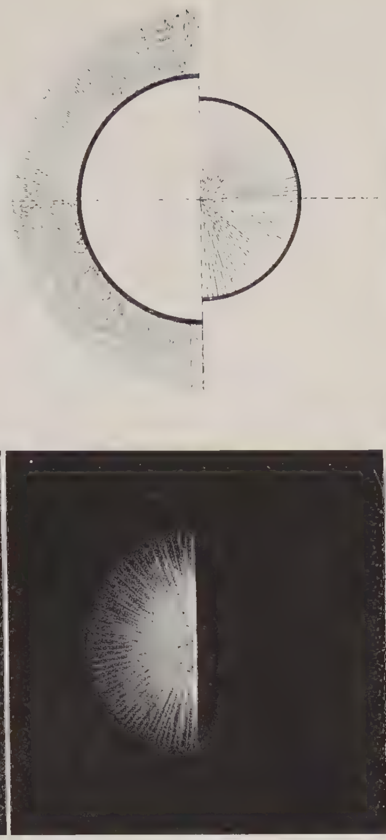
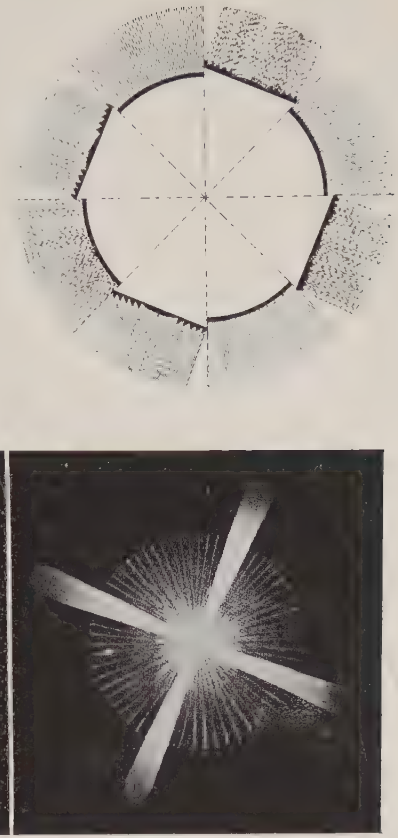


Figs. 288.—Fixed Light of 360°



Figs. 289.—Fixed Light of 180° and Dioptric Mirror of 180°



Figs. 290.—Fixed and Flashing Light. 4 flashing panels of 45° each; 4 fixed panels of 45° each

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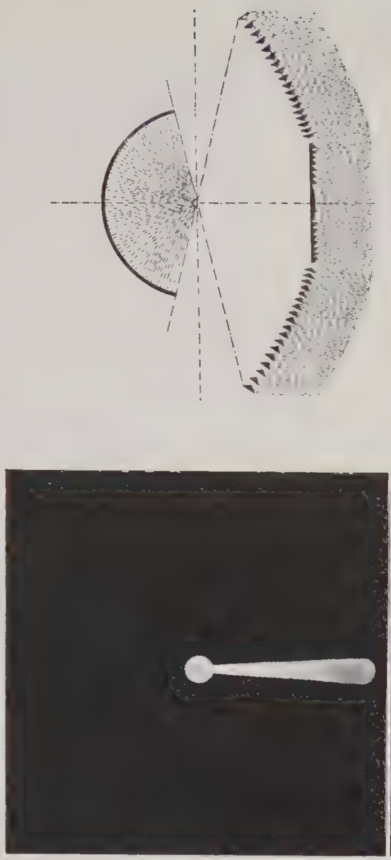
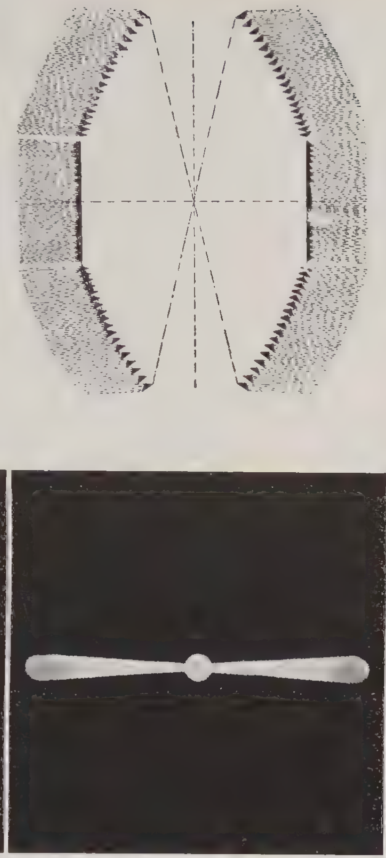
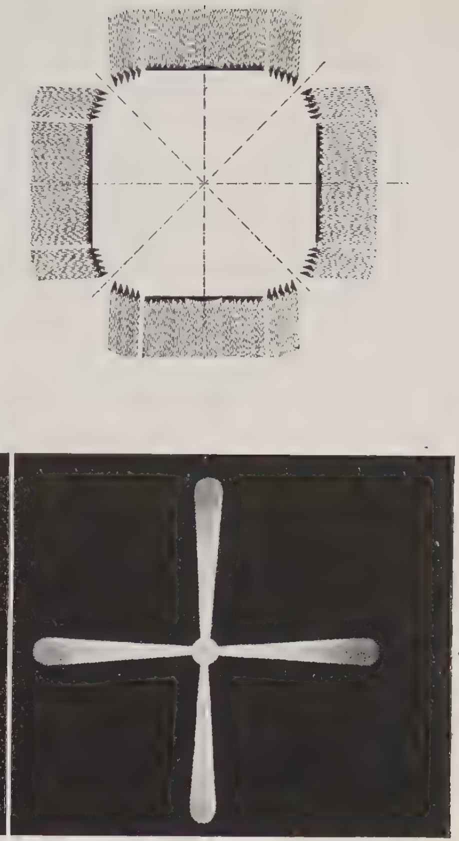


Figs. 291.—Single Flashing Light.
1 panel of 157°; Dioptric mirror 170°



Figs. 292.—Single Flashing Light.
2 panels of 157°



Figs. 293.—Single Flashing Light.
4 panels of 90°.

or rather, applied it in an elementary form in 1822. As then exemplified, it consisted of a plano-convex lens set vertically in front of the light, so that all rays passing through the lens were transmitted horizontally. To the central lens were then added a number of parallel lenses of triangular form, which served to refract a certain proportion of the rays passing above and below it. The amount of non-utilised light was still considerable.

Fresnel disposed his lenses so as to form a cylinder completely enclosing the light, which thus illuminated the entire circumference uniformly. Stevenson devised a variation known as the **holophotal** system, by which, in modern practice, the light is surrounded by a series of panels, each containing a circular central lens with annular adjuncts, the result being a concentration of the rays in a corresponding series of pencils, with intervening sectors of darkness.[1] Such an arrangement lends itself to the production of flash-lights by the revolution of the lenticular apparatus around the light, the illuminations being alternated with periods of obscurity, either total or partial, according as the apparatus is holophotal throughout, or combined with a fixed light.

With the object of still further strengthening the serviceable illumination, mirrors were placed above and below the lenses so as to reflect many of the rays which escaped the latter. This conjunction of the reflective and refractive principles led to the adoption of the term **catadioptric** to distinguish it. The mirrors were eventually replaced by lenticular or catadioptric prisms capable of effecting the same object, the rays entering the prisms being entirely reflected at one of the surfaces. Mirrors behind the light were similarly replaced. Altogether, a very high percentage of the total illumination is utilised, the vertical angle of Fresnel lenses now reaching 80° and, in some cases, 92°. The upper and lower prisms have, in many instances, been suppressed.

Yet another application of lens concentration is to be found in the **Azimuthal Condensing System** of Thomas Stevenson, in which holophotes and vertical prisms are employed to concentrate the light in special horizontal directions. This is exemplified in the apparatus constructed in connection with the Oronsay Lighthouse, where the light, as a leading light, is required to be seen along two intersecting axes of unequal length. in the one case for a distance of 15 miles, and in the other case for a distance of 7 miles. The dark or landward sector embraces an arc of rather more than 180 degrees.

A curious and interesting application of ray deflection was at one time applied at the Arnish Rock in the Hebrides. The lighthouse is situated on a rock separated from the Island of Lewis by a channel 500 feet in width. It contained no source of illumination itself, but received on a mirror a pencil

[1] Strictly, a Holophote is an arrangement by which the which of the rays are collected and transmitted in a single direction, but the term is generally, though loosely, applied to any annular lens or panel

of light rays from a lighthouse on Lewis. This ray was then deflected by prisms to pass onward in the direction required for the purposes of navigation. The arrangement has since been superseded by a gas beacon.

As at present constructed, **a modern lenticular panel** consists of a central, circular, plano-convex lens with annular adjuncts, and upper and lower catadioptric elements, the whole set in a gun-metal frame. If used for regular flashes, the optical apparatus will commonly be divided into four panels, each comprising a luminous angle of 90 degrees; but the number of panels may be decreased or augmented at will. Thus there may be six panels of 60 degrees or eight of 45 degrees, and so on. Yet it must be borne in mind that with an increase in the number of panels, there is a corresponding decrease in the intensity of the light. The beam of maximum power is attained by a single panel of about 180 degrees, with a lenticular mirror [1] behind the light capable of reflecting to the focus all rays impinging upon it. Apparatus of this concentration calls for very rapid rotation, such as would be incompatible with the old system of revolution on rollers. The required rotation is actually achieved by supporting the column or pedestal of the lenticular apparatus in a bath of mercury, which very materially reduces the friction of movement. A column so supported is known as a mercury float pedestal.

The biform, triform, and quadriform arrangements of superimposed lights, depending as they do upon the increase in total illumination instead of upon the unit intensity, have more or less ceased to be generally utilised. They are mainly serviceable in misty weather, the separate burners of which they are composed being individually ignitable and extinguishable at will.[2]

Lighthouse Orders.—For the purpose of classification, the lights of lighthouses are divided into a series of orders, according to their focal distances,[3] as follows :—

Order.	Focal distance in millimetres.
Hyper-radial,	1,330
Meso-radial,	1,125
First order,	920
Second order,	700
Third order,	500
,, small type,	375
Fourth order,	250
Fifth order,	187·5
Sixth order,	150

[1] A lenticular mirror is superior to a metallic mirror in regard to accuracy and durability. Large metal mirrors do not always retain their shape, and lose some of their brilliancy through the scratching of the surface from continued rubbing.

[2] Several *appareils jumeaux* (twin apparatus) are in use in the French Lighthouse Service. The lights are placed side by side instead of being superimposed, and send out parallel beams.

[3] The focal distance is the distance from the vertical axis of the light to the lens, or one-half the diameter of the apparatus on the focal plane.

[*To face p.* 358.

Fig. 294.—Fourth Order Red and White Single Flashing Apparatus, 250 mm. Focal Distance, with 7 feet diameter Lantern for "Bull" Lightship on River Humber (Chance Bros. & Co., Ltd.).

Lighthouse at Tyne North Pier.[1]—The lighthouse stands on a cylindrical base, 30 feet in diameter and 6 feet 9 inches high, composed of four courses of axed work, with rustic bed-joints. Immediately above the base are two moulded courses, from the top of which the rock-faced shaft rises. The diameter at the bottom of the rock-faced work is 25 feet and at the top 17 feet 6 inches, which, as the internal diameter is 12 feet from top to bottom, gives a thickness of wall of 9 feet in the base, 6 feet 6 inches at the bottom of the shaft proper, and 2 feet 9 inches at the top. The profile of the shaft is curved with a radius of 144 feet, and it is surmounted by two axed courses corbelled out to form a gallery round the lantern. The structure is faced with Cornish granite throughout, and, except above the floor of the service-room, where the walls, being thin, are of double-faced stones, the whole is backed with concrete. Windows are provided, one in the service-room and two at different levels below. The service room is approached by a winding staircase of York stone having a width of 2 feet 6 inches.

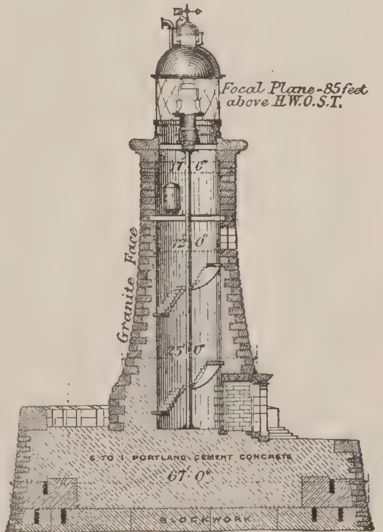


Fig. 295.—Lighthouse at Tyne North Pier.

The optical apparatus is triple flashing of the third order, consisting of two groups of lenses, each group having three panels. The focal plane is 85 feet above high-water level. The base is of cast-iron and circular; it is supported on a mercury pedestal and revolves once in 20 seconds. The machine will run for four hours without rewinding, and the power absorbed is 5,000 foot-lbs. per hour. The luminary is a single mantle "Matthews" incandescent oil burner. The intensity of the beam is 38,000 candle-power. The character of the light is triple flashing, white, every ten seconds. The whole of the incandescent system is in duplicate, and, as a further precaution, wick burners are provided as a stand-by.

The lighthouse is equipped with a fog signal, which is sounded from two similar horns, projecting through the copper cupola and pointing in two different directions. Each horn can be withdrawn from action for the purpose of replacing a reed or making any adjustment without interfering with the other. The pressure at which the signal is blown is 15 lbs. per square inch, and the note is B flat. The air for sounding the signal is compressed by one or other of two vertical compressors in the engine-room, which is situated below the promenade round the pierhead and in no part directly

[1] Barling on the Tyne North Pier, *Min. Proc. Inst. C.E.*, vol. clxxx.

under the lighthouse. Each compressor is provided with two tandem cylinders, one above the other, the pistons being of the trunk type. The upper cylinders are $4\frac{1}{2}$ inches diameter and the lower 8 inches, the stroke being 6 inches. The machines work with a 4 B.H.P. electric motor at 280 revolutions per minute, and at this speed are capable of supplying 15 cubic feet of air per minute at a pressure of 20 lbs. per square inch. Current is continuous at 430 volts. There is an oil engine in reserve.

The suction valves communicating with the annular space between the trunk of the upper cylinder and the walls of the lower can be put out of action when the compressor works, thus reducing the output but proportionately increasing the pressure. This arrangement renders it possible to have a reserve store of air compressed to 100 lbs. per square inch for immediate action in the event of sudden fog. The capacity of the two receivers in which the air is stored is 132 cubic feet, thus giving 440 cubic feet at a pressure of 20 lbs. above that of the atmosphere, or a supply sufficient to maintain the signal in full blast for nearly half an hour.

The blowing receiver in the service-room has a capacity of 30 cubic feet, and is coupled through the intermediary of a reducing valve with the storage receivers in the engine-room. The actuating valve is operated from the clock, which rotates the optical apparatus. The service pipes are of copper, $2\frac{1}{2}$ inches in diameter. When in action, the reed-horn gives a 3-second blast every ten seconds.

Range of Light.—The distance penetrable by rays of light varies obviously with the transparency or opacity of the atmosphere. If the medium were a vacuum, the range would be proportional to the square root of the luminous intensity. This relationship, however, cannot be realised in an atmosphere such as envelops the British coasts: often foggy, only occasionally very clear. And as localities vary in their meteorological experiences, no serviceable standard can be devised. The French have a system of denoting the visible range of their lights during two percentages of the whole year —viz., 50 per cent. corresponding to clear weather only, and 90 per cent. corresponding to the inclusion of moderately misty and variable weather. It is difficult to see how such a system could be successfully applied to British pharology, where climatic changes are more frequent and much more adverse.

The table on p. 361 gives the nominal optical range in nautical miles for white light of various candle-powers in Hefner units, but, for the reason stated, its practical utility is doubtful.[1]

Powerful lights in clear weather may easily exceed their geographical range—*i.e.*, the distance at which, owing to the earth's rotundity, they cease to reach the eye of the observer. This distance varies with the respective heights of the light and of the observer, and also with the degree of latitude.

[1] For a valuation of units, see footnote, p. 350.

Candle Power. (Hefner.)	Range. Clear Weather.	Range. Misty.	Candle Power. (Hefner.)	Range. Clear Weather.	Range. Misty.
	Miles.	Miles.		Miles.	Miles.
1	1·5	1·2	2,500	16·9	8·5
5	2·9	2·1	5,000	19·2	9·4
10	3·7	2·6	10,000	21·6	10·3
15	4·3	2·9	25,000	24·9	11·6
20	4·8	3·2	50,000	27·6	12·6
50	6·4	4·0	100,000	30·2	13·6
100	7·9	4·7	250,000	33·8	14·9
250	10·1	5·7	500,000	36·8	16·0
500	12·0	6·5	1,000,000	39·6	17·0
1,000	14·0	7·3			

The latter affects the radius of curvature (R); but, assuming it to be known as also the levels (H and h) of the source of light and of the station of observation, the limiting distance D is given by the formula

$$D = k \,.\, \sqrt{2\,R}\,(\sqrt{H} + \sqrt{h}),$$

where k is a coefficient representing the effect of atmospheric refraction.

Taking the earth's radius as roughly 4,000 miles, a convenient approximation with D in miles, H and h in feet, is

$$D = k \,.\, \tfrac{6}{5}\{\sqrt{H} + \sqrt{h}\}.$$

Neglecting atmospheric refraction, and taking the case of an observer standing on the deck of a vessel with his line of sight 15 feet above the water level, the following is a short table of distances of visibility for lights at various heights, assuming, of course, that the intensity or candle-power is adequate :—

Height of Light above Sea Level in Feet.	Distance of Visibility in Miles.	Height of Light above Sea Level in Feet.	Distance of Visibility in Miles.
5,	7	50,	12·5
10,	8	75,	14·4
15,	8·9	100,	16
20,	9·5	200,	20·7
25,	10·2	300,	24·3
30,	10·7	400,	27·4
35,	11·2	500,	30
40,	11·7	1,000,	40·7

Identification of Light Signals.—For the purpose of identification, various characteristics are conferred upon lights. Formerly colours were largely relied upon, but the great difference in range of the three chief varieties of light—viz., white, red, and green—militates very much against the efficiency of the method. Red cannot be seen at half the distance penetrable by white light, and green is even less powerful. At a distance of two miles, a white light of 3 candle-power is readily discernible, while from 30 to 40 candle-power would be requisite to bring a red or green light into equal

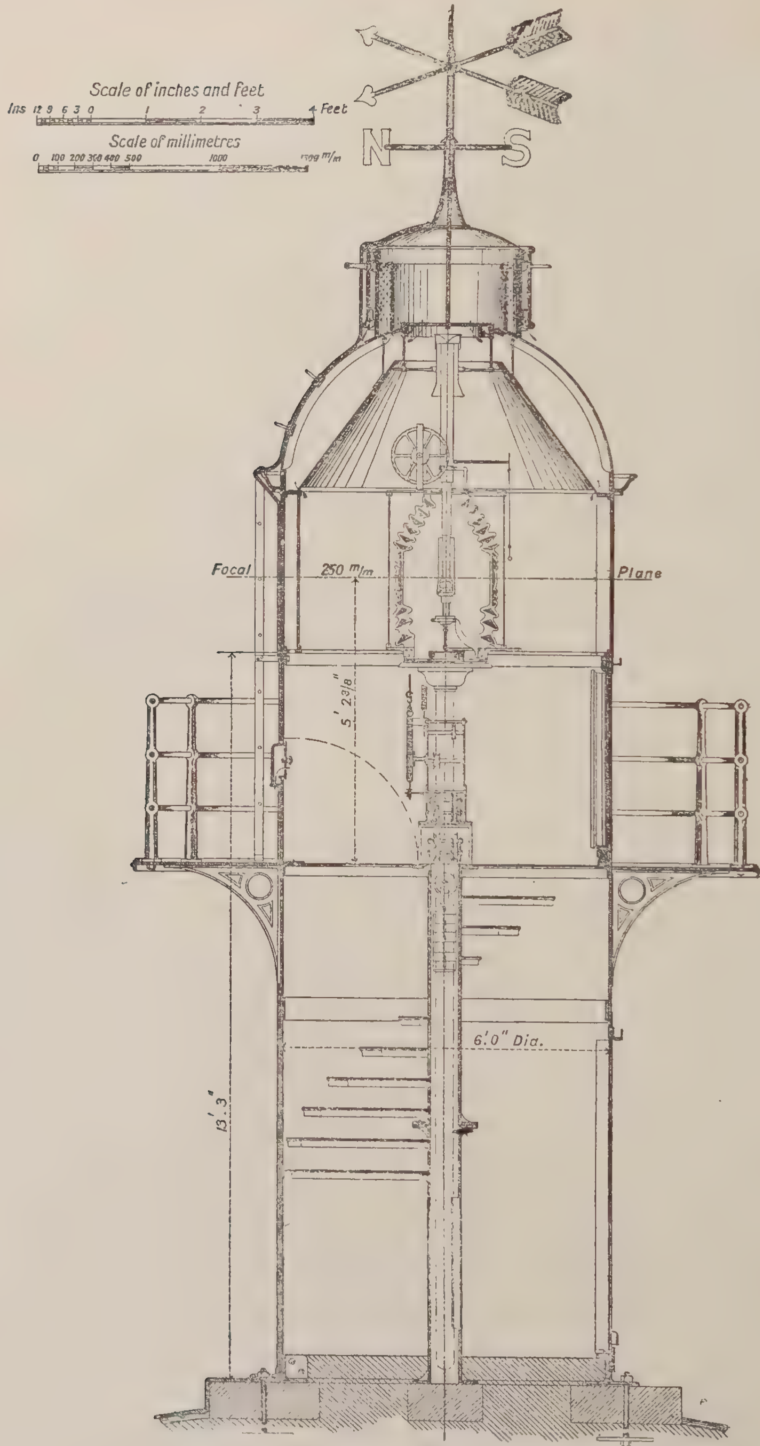


Fig. 296.—Tower Lantern and Fourth Order Occulting Light.

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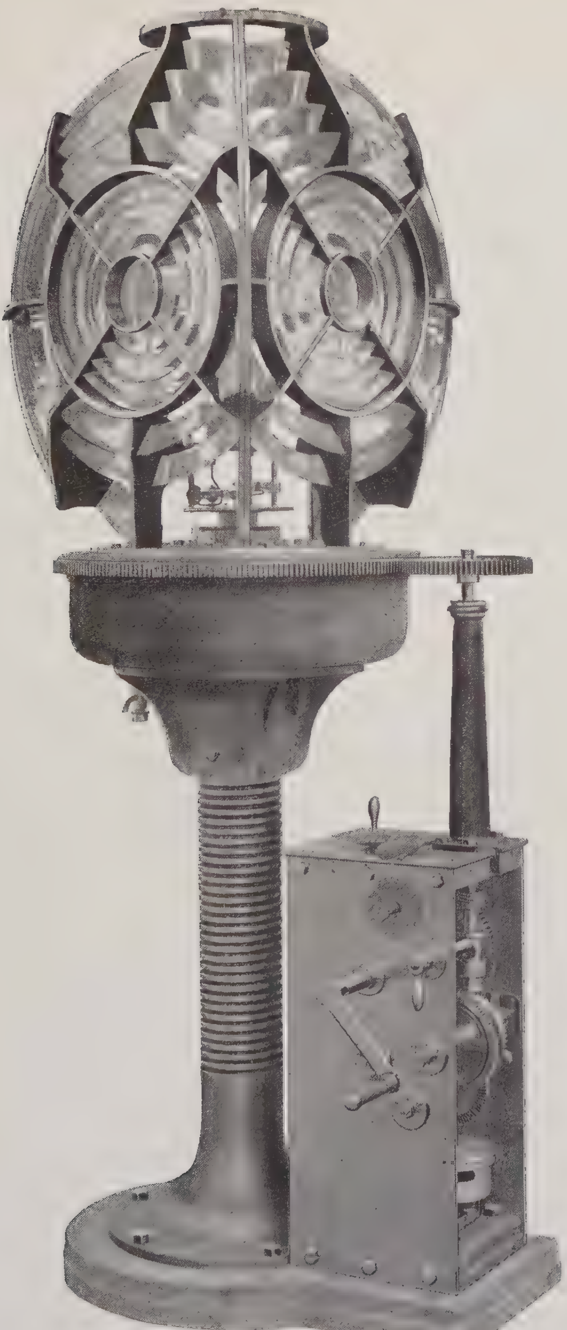

Fig. 297.—Fourth Order Double Flashing Light, 250 mm. Focal Distance, for Vigil Lighthouse, St. Lucia (Chance Bros. & Co., Ltd.).

prominence. Moreover, there was not much scope for variation with merely three alternatives.

With the introduction of the **group flashing system,** devised by Dr. Hopkinson in 1875, a new and preferable means of identification came into vogue, and its utility has been still further extended by the introduction of **lightning flash-lights** under the inspiration of M. Bourdelles. The first consists of a definite series of illuminations and eclipses, variable in extent and sequence. The latter derives its title from the extreme rapidity of its appearance and disappearance, the period of visibility being the minimum required for imprinting a distinct visual impression. Further exposure is now found to be unnecessary, as it is covered by the persistence of the image on the retina. Light rays, accordingly, instead of being uselessly expended in emphasising their effect on one point, may be deflected to another, with much more serviceable results. The principle, in fact, is that of a highly concentrated beam rotating rapidly and reappearing at intervals of a few seconds. The duration of the flash, though short, is ample for recognition, and its frequent appearance, besides affording greater scope for characterisation, enables the mariner to verify his position with greater assurance than was feasible with an arrangement of slowly moving lights. It was not impossible, of course, to provide a considerable number of flashes under the system of rotation on rollers, but it could only be done by increasing the number of lens panels at the expense of luminous power; whereas, by the introduction of mercury flotation, a much greater rotary velocity may be imparted to the apparatus, with less effort and friction. In some cases of old lenticular apparatus, the number of panels is as high as twenty-four. The diffusion of light, therefore, is very great, and the beams suffer correspondingly in intensity and penetration.

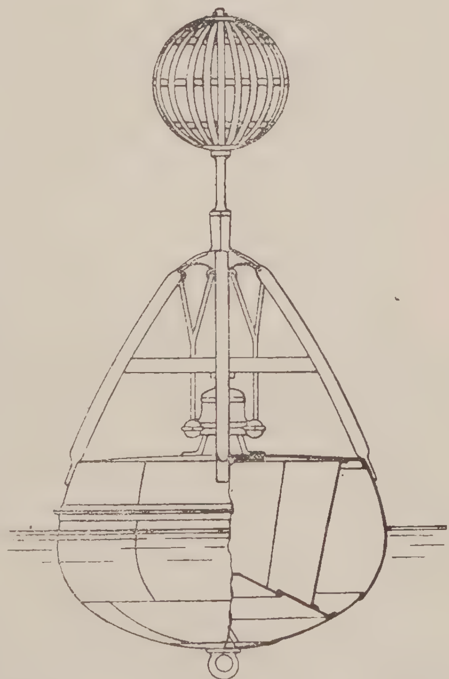

Fig. 298.—Bell Buoy used by the Corporation of Trinity House.

The minimum duration of a visible flash has been determined by laboratory experiments as one-tenth of a second; but under conditions obtaining in marine navigation, the period will be increased to about one-fifth of a second for white lights and rather more for coloured lights.

Flashes are either uniformly regular or arranged in groups of two, three, or more. The interval between successive flashes varies from about $2\frac{1}{2}$ to 30 seconds according to locality. In grouped flashes, short eclipses of $1\frac{1}{2}$ to 4 seconds are common. Certain lighthouses signal a definite number, as, for example, that at Minot's Ledge, U.S.A., which constantly repeats the figures 143 in a series of flashes separated from each other in the same group by an interval of 2 seconds, the groups being separated by an interval of 3 seconds, and the entire signal followed by an eclipse of 15 seconds. The whole period covers 30 seconds, which is the time of a single revolution of the apparatus.

Sound Signals.—Lighting, while effective enough as a guiding agency in darkness, is practically useless in fog, and reliance has then to be placed upon sound as a warning medium. This is accomplished in various ways, principally by foghorns and explosives on shore, or on lightships, and also at sea by bell-buoys and whistling buoys. Of foghorns, it need simply be said that they consist of a trumpet fitted with a siren, reed, or diaphone, through which a strong blast of compressed air or steam is expelled at definite intervals. They are raucous and unmusical in the extreme. The siren gives out a note of high frequency due to the impulsion of air or steam through a series of holes in a rapidly revolving disc.

Fog Siren Plant on Lightship "Alarm."—The following particulars relating to the fog siren plant on the lightship "Alarm" are extracted from the paper by Lieut. Gracey, already referred to :—[1]

The apparatus, which was supplied by Messrs. Chance, of Birmingham, is designed to give a blast of 2 seconds' duration every 20 seconds. The air compressing plant is in duplicate, each unit being of sufficient capacity to sound the signal alone, the other being available in case of breakdown.

Each set consists of a Hornsby 11 B.H.P. horizontal oil engine, coupled direct to a double-acting, single-stage compressor, and mounted on a combination bedplate. The engine and compressor cylinders are cooled by sea water, circulated by a pump driven off the crankshaft. The compressors discharge into two large storage receivers, each of which has a capacity of 100 cubic feet, and is always kept charged at a pressure of 100 lbs. per square inch. This supply is sufficient to sound the fog signal for about 15 minutes, which allows ample time to start one of the engines. The engines are started by means of compressed air from the storage receivers, but a hand pump and container are provided in case this supply is not available.

The compressed air is led through steel pipes of large diameter to the sounding receiver, and enters it through a reducing valve at a pressure of 25 lbs. per square inch. The siren is 5 inches in diameter, and gives a note of 182 vibrations per second, which corresponds to upper F sharp in the bass clef. The admission of air to the siren is regulated by a pilot valve, operated by a special weight driven clock with a governor, which maintains the accuracy

[1] *Vide* p. 344.

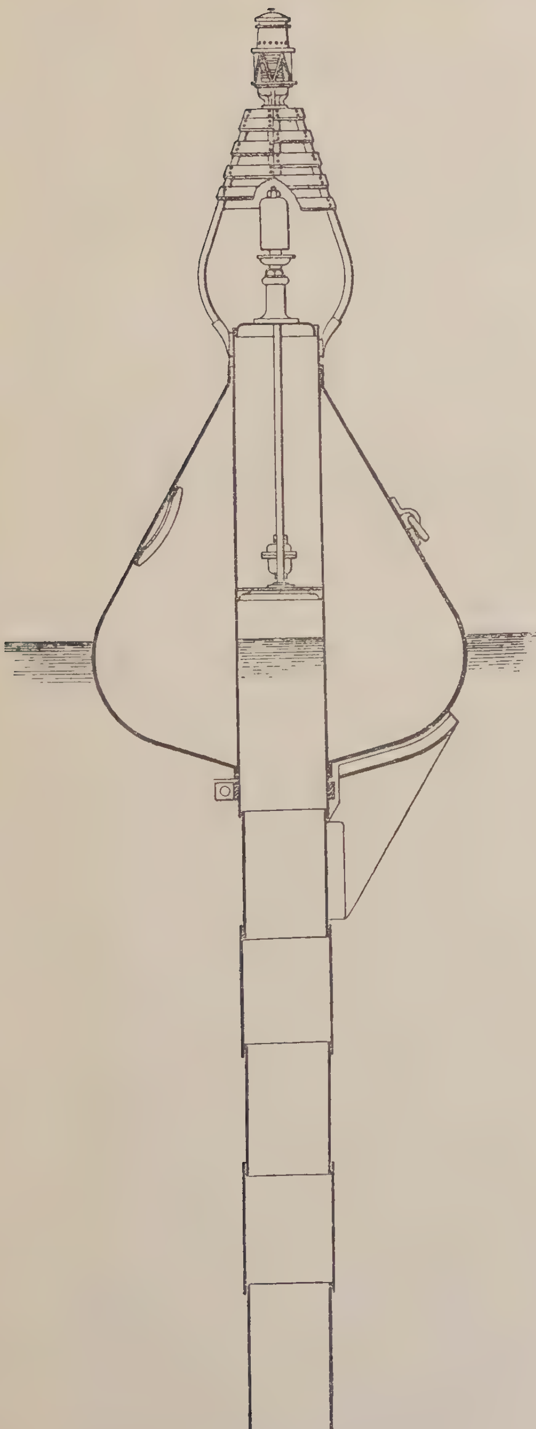

Fig. 299.—Courtenay Whistling Buoy.

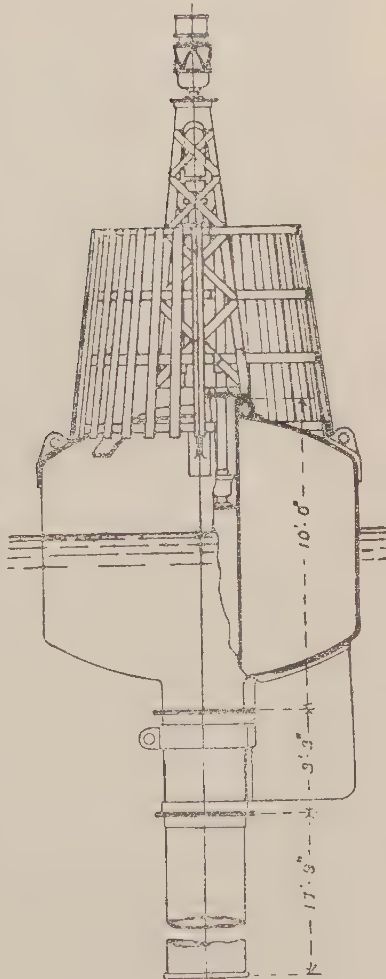

Fig. 300.—Whistling Gas Buoy.

of the sounding periods. The trumpet has a copper mushroom head 4 feet in diameter, and stands 14 feet above the top of the deckhouse. An autographic recording instrument is fitted for registering the time and duration of the blasts.

Of **bell-buoys** there are several different types. A fixed bell may be struck by pendant clappers, or by a set of balls rolling freely in horizontal grooves or cylinders. The former is the more general arrangement. When the water is smooth, as is commonly the case in foggy weather, neither of these appliances can be counted upon to emit signals, depending as they do upon the swaying action of waves. In that event, automatic apparatus, worked by the agency of oil gas, as in the Pintsch system, or of carbonic acid gas, as in the Dalèn system, has been found useful. The gas forces up a diaphragm until it works a lever which closes the inlet valve and opens the outlet, simultaneously actuating another lever, which, in conjunction with a strong spring, impels the hammer against the rim of the bell.

Whistling buoys are actuated by the rise and fall of the buoy in a swell. In the Courtenay type, air is drawn into a long central tube during the period of rising. The entrance is controlled by a valve, and when the buoy descends the imprisoned air is expelled through the whistle, emitting a penetrating sound of no particular musical value. Another type of whistling buoy has double tubes. Whistling buoys are best adapted to fairly deep water where there is an almost constant swell.

Sound, however, in air is but an imperfect medium for the notification of danger. It gives no reliable indication, and indeed often conveys a very misleading impression as to locality. Zones of silence are found to lie within the sonorous area.[1] Yet, until fogs are dissipable by human agency, it is difficult to see what other means could be universally substituted ; and, certainly, the system is reliable in so far as it signifies the imminence of danger, though in many cases the exact location of the warning is a matter of conjecture.

Subaqueous Signals.—In water, the sense of direction by sound is more determinable. A system of submarine signalling through the agency of a bell struck by a clapper at depths varying from 10 feet to 30 feet below the surface of the water, has within recent years been promoted by the Submarine Signalling Company. The sound is transmitted through the water to receivers fixed low in the ship's sides, and thence through microphones to electric indicators, with results which are very satisfactory so long as the instruments are immersed to depths of not less than 10 feet and preferably of about 25 feet. The distance traversed has reached 8, 10, and even 15 miles. By turning the ship in various directions, the quarter from which the sound emanates can be easily determined as the sound waves do

[1] In some interesting experiments carried out off the Isle of Wight, fog sound-signals could not be heard at all at 2 miles from the coast, although they were distinctly audible 10 miles out and were heard again at half a mile from land.

not appreciably affect the receiver unless it faces the direction from which they come. There is manifestly much scope for the development of this principle of sound transmission. The chief difficulty hitherto has been that of ensuring a constant and regular striking of the warning bell without the necessity for human attendance. In this respect, the Dalèn automatic striking action, already alluded to, promises results of a satisfactory nature.

At present there are three principal systems in vogue: viz., (1) Bells suspended from lightships and struck by the agency of compressed air controlled by a code-ringing device in the engine-room; (2) bells supported by buoys and worked by the aid of discs, acting on the principle of a sea anchor, so that the difference in movement between the disc and buoy operates delicate mechanism; and (3) bells supported on tripods resting on the sea floor, and having electrical communication with a shore station.

The following is the description of an automatic bell as devised by the Submarine Signalling Company :—

The power has to be obtained from the rise and fall of the sea surface, and the main difficulty is to ensure that each blow of the bell shall be of unvarying intensity. A vane or fan is made use of to actuate the internal mechanism as it rises and falls. The mechanism is suspended to a special can-buoy, but may be attached to any form of buoy having sufficient buoyancy to support it. To the bottom of the buoy is rivetted a rigid framework extending to a depth of about 18 feet, and the mechanism is protected from the mooring chain by being enclosed within a hollow receptacle 4 feet square by 5 feet long, made of boiler plate, open at the top and bottom. The striking gear, which is enclosed in a water-tight case, comprises a balanced vane, normally horizontal, attached to a rocking shaft, passing through two glands to the interior of the case and having two spur wheels, and a crankshaft with two loosely mounted spur wheels attached to two rocking arms, each of the latter carrying a pair of pawls; one pair of spur wheels is in direct mesh one with the other, whilst the other pair is in mesh with an idler, thus causing the rocking arms to travel in opposite directions.

A wave, raising the buoy, will cause the vane to turn through an angle in a downward direction, which, by means of one pair of pawls, starts turning the crankshaft and connecting-rod over the top centre and compressing the spring; a corresponding downward movement of the buoy causes the vane to turn through an angle in an upward direction, and by means of the idler spur wheels and the other pair of pawls, causes the crankshaft to turn further in its original direction, thus further compressing the spring. The vane can assume a vertical position either up or down, so that there is no shock should the buoy plunge heavily in rough weather. Half a turn of the crankshaft compresses the spring to the limit and brings the hammer right back. The next movement of the vane carries the crank-pin past the bottom centre, when the spring comes into action, causing the crank to turn the balance of one revolution and a smart blow is then struck on the bell.

The mechanism is cumulative. One slight movement of the vane may not be sufficient to cause the bell to ring, but a sequence of such movements will do so, as each performs some work on the spring. In an ordinary sea the bell will ring about 15 blows a minute, and observations show that on

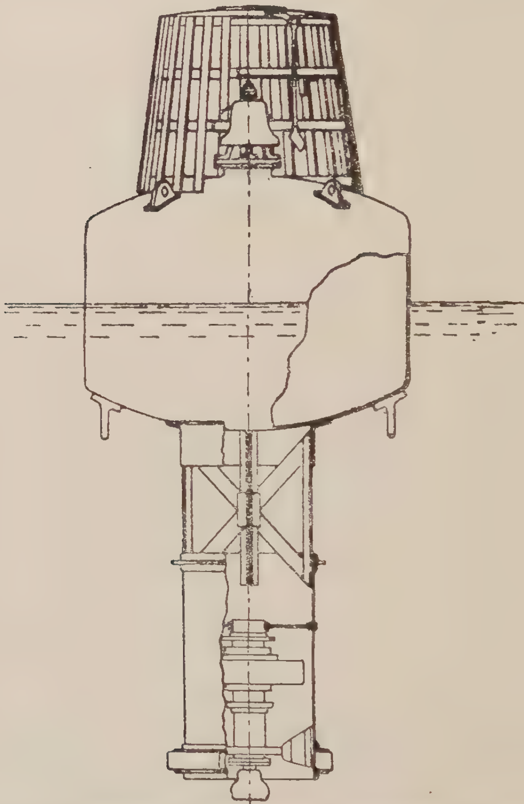

Fig. 301.—Submarine Bell-buoy used by the Corporation of Trinity House.

the ocean, even in an apparent calm, there is always sufficient movement to ring the bell several times a minute.

The bell used is one having a sound-bow of about 15 inches in diameter, and weighing 220 lbs.

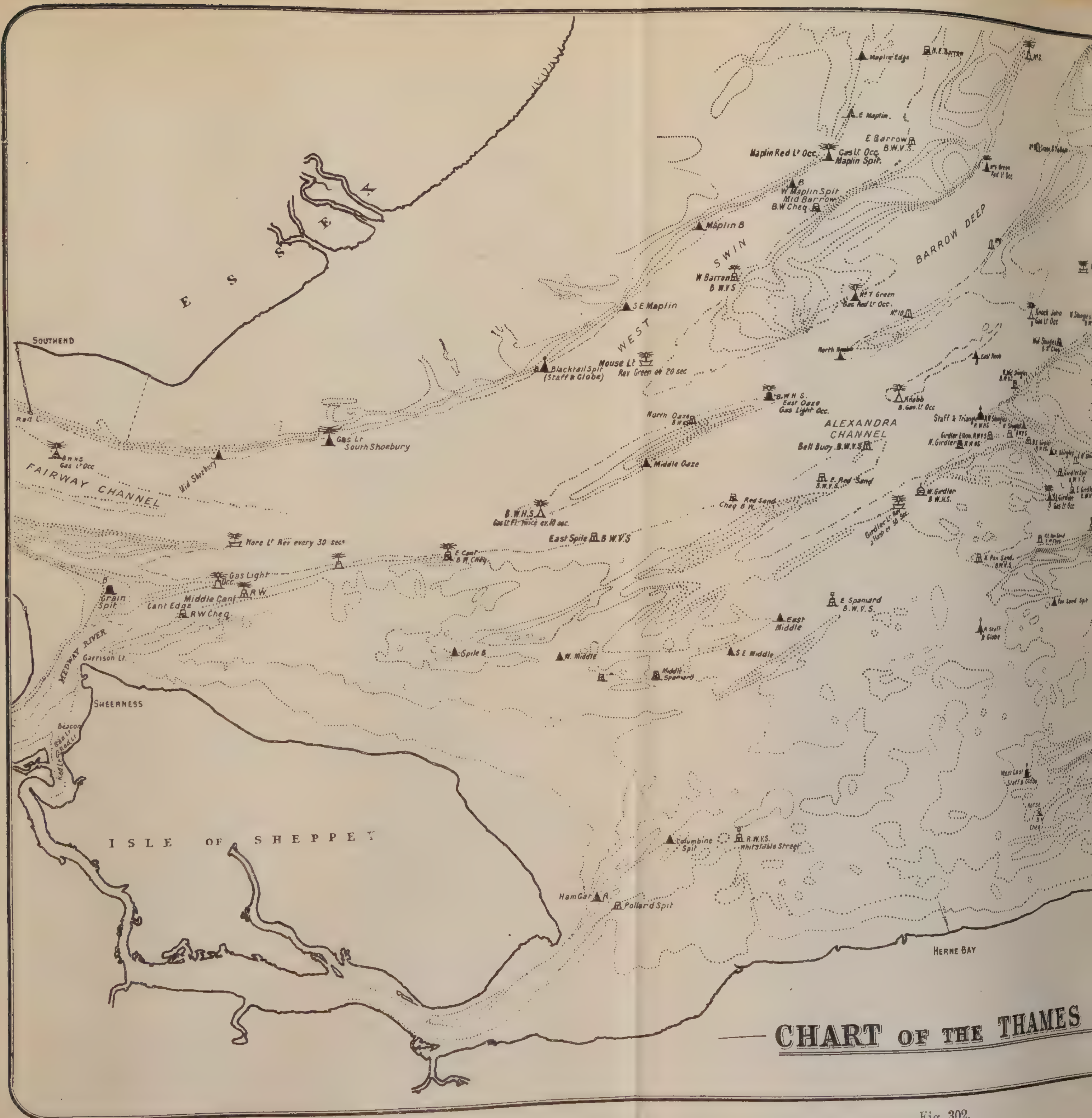

CHART OF THE THAMES

Fig. 302.

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Bell and Bain, Limited, Printers, Glasgow.